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THE ASSOCIATION was organized in Boston, Mass., on June 21, 1882, with the object of providing its members with means of social intercourse and for the exchange of knowledge pertaining to the construction and management of water works. From an original membership of only TWENTY-SEVEN, its growth has prospered until now it includes the names of 800 men. Its membership is divided into six classes, viz.:

A Member shall be an officer or employee of a public or private water works, an engineer, chemist or other person qualified to aid or interested in the advancement of knowledge relative to water works.

An Honorary Member shall be a person of acknowledged eminence in some branch of water supply or of engineering.

A Life Member shall be a member whose service to the Association entitles him to special recognition by the Association.

A Junior shall be not less than eighteen years nor more than twenty-five years of age, a student or connected with water supply work.

An Associate shall be either a person, firm or corporation engaged in manufacturing or furnishing materials or supplies for the construction or maintenance of water works.

A Corporate Member shall be either a Water Board, Commission, Company or Municipal Corporation.

The initiation fees and annual dues are as follows:

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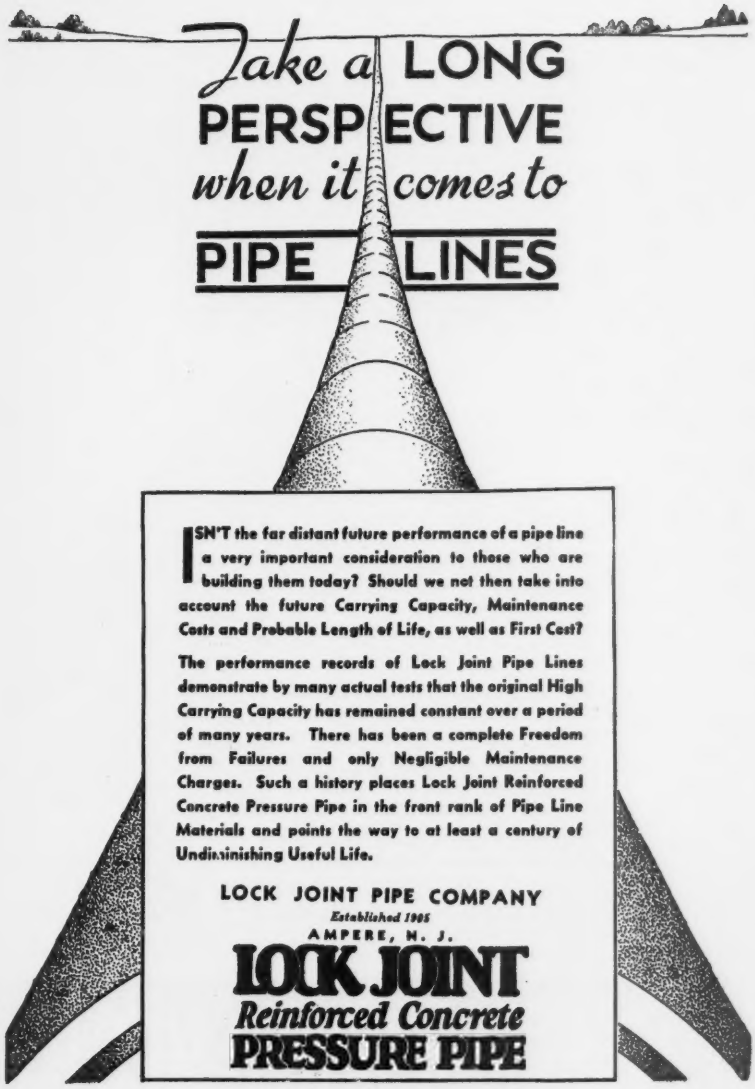
This Association has at least eight regular meetings each year, of which five are held in Boston, one in northern New England, one in southern New England, and one, the annual convention, held in September on such date as the Executive Committee may designate.



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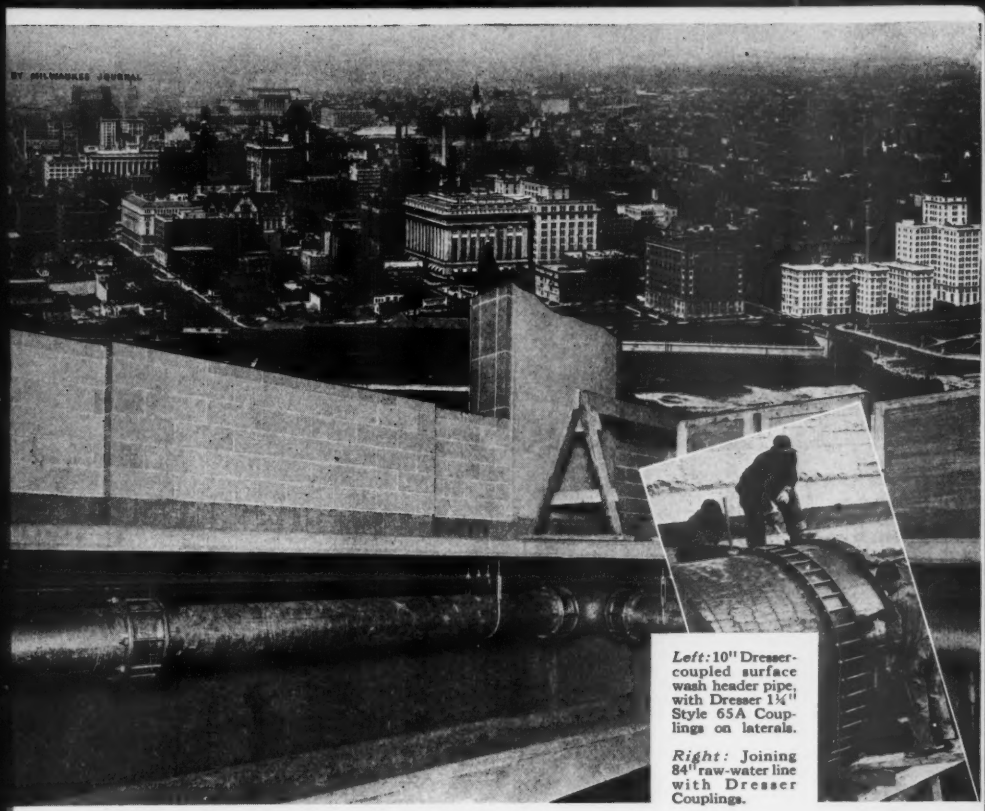

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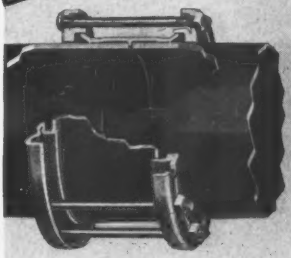

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Table of Contents

PAGE

Economics of Water Purification. By Arthur L. Shaw and E. Sherman Chase 131

The Need for Improvements in the Distribution System of the Metropolitan Water District. By Eugene C. Hultman 151

Methods Used to Finance Main Pipe Extensions in the Hartford, Connecticut, Distribution System. By Warren A. Gentner 178

The New Sand Wash Brook Dam and Reservoir at Pittsfield, Massachusetts. By Samuel M. Ellsworth 190

Experiences with Reinforced Concrete Standpipes. By Charles W. Sherman 201

Economics of Pumping Equipment. By D. D. Eames 214

Diesel Pumping Plant at Milford, Massachusetts By M. H. Goff 225

Principles of Corrosion and Its Prevention. By Frank N. Speller 228

Cathodic Protection. By A. V. Smith 233

Enforcing Payment of Water Bills. By Daniel M. Sullivan 244

New Developments in Fluid Meters that May Affect Water Meter Design. By Walter Parker 248

New Developments in the Chemistry of Rubber and Like Substances in Relation to Water Works Practice. By E. A. Hauser 254

Proceedings:

March Meeting 256

April Meeting 257

May Meeting 257

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ECONOMICS OF WATER PURIFICATION

BY ARTHUR L. SHAW AND E. SHERMAN CHASE*

[*Read September 21, 1937.*]

The economics of water purification is a topic which may be discussed from two standpoints; first, that of the broad economic value of pure water; and second, that of the actual cost of construction and operation of the works necessary to produce water of safe and satisfactory quality. This paper attempts to discuss the subject from both standpoints.

Economic Value of Pure Water

Definition of "Pure Water". In his book, "The Value of Pure Water," the late George C. Whipple discusses at some length the meaning of the term "pure and wholesome water." Summarizing his discussion in the light of present standards and referring of course to public water supplies, it may be stated that a pure water is free from poisonous substances, disease-producing bacteria and bacteria indicative of pollution, and that this water is clear, colorless, odorless and tasteless, low in hardness, mineral matter, iron and manganese and relatively non-corrosive.[1]

Few waters can qualify as possessing all these desirable characteristics. It is the object of water purification to produce waters that can qualify.

Whipple attempted to appraise the economic benefit arising from pure water by estimating in dollars and cents the depreciation in value that results from different kinds and degrees of impairment in quality. In light of the public demand for water of better quality than ever before and with the definition of pure water in mind, we also have attempted a tentative economic appraisal of certain quality characteristics of water supplies.

Health Value. While it is obvious today that a public water supply should be free from disease germs, it is not so long ago that

*Partners, Metcalf & Eddy, Boston, Massachusetts.

[1]See also, Drinking Water Standards adopted by the Treasury Department, June 20, 1925. Public Health Reports, Reprint 1029.

the causation of typhoid fever, for example, by polluted water was in dispute. Nevertheless, the protection of the public health has been generally the first consideration leading to the adoption of water purification processes.

The appraisal of the value of the health protection afforded by the purification of a water supply is a somewhat cold-blooded procedure. It is, however, a much more cold-blooded matter to neglect those precautions which make for a uniformly safe supply in order to effect an apparent economy in municipal administration. Furthermore, such neglect in a number of cases has proved to be false economy where substantial monetary damages have been awarded for deaths and illness due to infected water carelessly furnished by municipalities.

An instance of heavy damages for waterborne typhoid paid by a city was described to This Association by Earl Devendorf on September 21, 1933[2] Devendorf referred to an outbreak of waterborne typhoid in the city of Olean, New York, in 1928, in which 21 deaths occurred among 238 cases of the disease, following which and up to the beginning of 1933, $425,000 had been paid out for claims resulting from the outbreak. If the Olean experience can serve as a yardstick, any municipality responsible for waterborne typhoid is potentially liable for damages equivalent to about $20,000 per death.

We have no way of knowing with any degree of precision the effect of the general introduction of safe water supplies throughout the country. We do know that in 1900, with a very small proportion of American cities supplied with filtered water and before the use of chlorination, the typhoid death rate in the United States was 35.8 per 100,000 population. In 1935 with most water supplies either filtered, chlorinated or from safe ground water sources, the typhoid death rate was 2.8. Assuming that only one-half the reduction in typhoid may be attributed to the increased safety of water supplies, there are some 20,000 fewer annual deaths from typhoid than would be the case if the water supplies had remained as generally unsafe as in 1900. Applying the Olean yardsticks of value the annual saving due to pure water amounts to $400,000,000.

Not all the dangers to health involved in supplying water to a community are due to the character of water as received. There are other hazards incidental to the distribution of the water, such as cross-connections with polluted private supplies, distribution reservoirs open to pollution, faulty plumbing, and, very occasionally, the dissolving of lead from service or household pipes of that material. In order to obtain the full economic value of purification works, it

[2]This Journal. 47, 298 (Sept. 1933).

is necessary therefore to take precautions against the introduction of pollution into the distribution system and against the production of a water of plumbo-solvent character. It is interesting to recall that in the Wolman-Gorman[3] 1931 study of waterborne typhoid fever, pollution through cross-connection ranked second on the basis of number of outbreaks and fourth on the basis of number of persons affected during the decade 1920-29.

Another point not to be overlooked is the fact that improvement in the safety of the water supply of one municipality not only reduces the waterborne disease in that particular community but also reduces the spread of such disease to other communities. No one can estimate the number of deaths from typhoid or the frequency of secondary but remote epidemics resulting about the beginning of this century from infection of transient visitors to large cities such as Philadelphia, Pittsburgh and St. Louis which were then supplied with dangerously polluted waters. Neither can one even guess as to the number of honeymoons to Niagara Falls, prior to the introduction of filtered water, that ended tragically as the result of typhoid contracted through the medium of the unpurified Niagara River supply. The outbreak of amoebic dysentery at Chicago during the Century of Progress Fair is a more recent example of how impure water of one city may affect disastrously the citizens of other communities.

Returning to the problem of expressing the value of pure water in monetary terms, we may assume that the average annual saving in typhoid deaths in any one community due to pure water is 15 per 100,000 population, equivalent to $300,000 annually on the basis of the Olean settlements. With the average water consumption approximating 100 g.p.d. per capita, this saving is equal to $30,000 per year for each million gallons of water supplied daily. In other words, if the sum of fixed and operating charges for water purification is not in excess of $30,000 annually per million gallons daily consumption, then the expenditure of this amount for purification is justified on the basis of economic saving to the community involved.

Value of Satisfactory Physical and Chemical Qualities. In view of the general safety of water supplies today it is perhaps of more interest to attempt the financial valuation of the physical and chemical qualities of a water supply. While it is far less important to furnish water of pleasing quality than to provide a safe water, high color and turbidity, disagreeable tastes and odors and rusty water are the cause of the customers' complaints.

It is well known that in municipalities having water supplies possessing qualities displeasing to the consumers much money is spent

[3]The significance of Water borne Typhoid Fever Outbreaks, 1920-1930, by Abel Wolman and Arthur E. Gorman.

for spring water for drinking purposes. The amounts spent depend upon the degree and character of the impairment. Disagreeable tastes and odors probably are responsible for the greatest expenditure. In New England we ordinarily think of taste and odors as due to microörganisms, but in other sections of the country industrial pollution of water supply streams is a factor to be taken into consideration. Furthermore, over-chlorination for precautionary reasons where the sources of supply are particularly hazardous, while safeguarding the safety of the supply, frequently results in objectionable chlorinous tastes and odors with much complaint on the part of the water users.

The late Morris Knowles, for many years a member of This Association, estimated several years ago that approximately 20 gal. of spring water per capita were purchased annually in Lawrence, Mass., as compared with 2 gal. in Boston, due to the unsatisfactory character of the supply of the former city. As ten cents per gallon is a reasonable figure for the cost of spring water delivered, it will be seen that supplies with objectionable characteristics may bring about annual expenditures for spring water ranging from twenty cents to two dollars per capita equivalent to about $2,000 to $20,000 per year per million gallons daily of water consumption. These figures represent the range in economically justified expenditures for annual charges for purification necessary to provide a water acceptable as regards tastes and odors.

To some extent, although not to the same degree as in the case of tastes and odors, high color and turbidity cause consumers to seek substitute supplies for public supplies possessing such characteristics. No data are available as to the extent to which color and turbidity cause the public to purchase bottled waters. As a matter of opinion only, it is not unlikely that a color of about 110 p.p.m. or a turbidity of 55 p.p.m. would result in the expenditure for spring water of an amount equivalent to that spent in Lawrence. Colors less than 10 p.p.m. or turbidities less than 5 p.p.m. are acceptable to the New England customer.

Assuming that the purchase of substitutes as the result of high color in the public supply varies directly as the color increases between 10 and 110 p.p.m., with the higher color bringing about the expenditure of $2.00 per capita yearly for spring water, then each part per million of color above 10 represents an economic loss of two cents per capita per year or $200 per year per million gallons daily of water consumption. For turbidity in excess of 5 p.p.m., the corresponding figures would be four cents and $400, respectively. To put it in another way, it may be stated that a municipality having a water supply with a color of 60 p.p.m. or a turbidity of 30 p.p.m.

would be economically justified in purifying this supply if the total annual charges did not exceed $10,000 per million gallons daily of consumption. These figures are based wholly upon the use of the supply for domestic purposes and do not take into account the economic effect of water quality upon many industrial processes.

In the case of waters of corrosive qualities, it is almost impossible to estimate the economic loss that results therefrom. An example, however, may serve to illustrate the very great losses that may be involved. A certain municipality has a water consumption of 850,000 g.p.d. Corrosion of the mains is so rapid, particularly after cleaning, that recleaning is necessary about every three years to maintain their capacity. Cleaning costs about 9 cents per linear foot and if done every three years means an average annual expenditure of 3 cents per foot. In this particular municipality there are over 100 miles of mains, which, if cleaned on the average of every three years, would result in an annual cleaning cost of about $16,000.

In addition to the effect of this corrosive water upon the public mains, there is serious impairment of service and household piping. There are about 3100 services in this town representing, in all probability, some $300,000 of value in inside and outside piping. It is conservative to estimate that the corrosive effect of the water cuts in two the useful life of these pipes. Assuming an average normal life of 20 years, the resulting loss from the shortened life of services and inside piping is $15,000 annually.

The total annual loss due to corrosive water in this town becomes then $31,000 equivalent to $36,500 per year per million gallons daily.

Gradations in degree of corrosiveness cannot be expressed quantitatively with accuracy. Furthermore local conditions such as length of mains, character of pipe and pipe linings used, and water consumption greatly affect the extent of loss due to corrosion. It is therefore impossible to suggest even a rough yardstick for measuring generally the economic loss resulting from corrosive water. Each case must be considered separately.

The loss due to hard water has been studied somewhat more than the losses from other characteristics and numerous articles have been published dealing with this matter. Whipple estimated that each part per million of hardness represented a cost for soap equal to 10 cents per million gallons or an annual cost of $36.50 per million gallons daily.

Some years ago the authors made a study of the cost to the water consumers of Dayton, Ohio, of using water having 350 p.p.m. of hardness instead of 100 p.p.m. such as could be obtained by means of a softening plant. Our estimates indicated that annual losses

due to excess soap and gas consumption and the cost of maintenance and depreciation of dual household supplies of soft water were $3.50 per capita, equivalent to about $150 per million gallons daily per part per million of excess hardness. In other cities with softer water the use of dual systems of soft water may not be so likely to prevail and the Dayton figure may be unduly high. However, H. W. Hudson[4] in the Journal of the American Water Works Association gives figures for excess costs of soap alone used in certain other cities with waters ranging in hardness from 70 to 555 p.p.m., and these costs indicate that the Dayton estimate was not too high when all the major harmful effects of hard water are taken into consideration.

While excessive amounts of iron and manganese are of definite economic significance it is particularly difficult to estimate in terms of money the losses involved. One method by which such an estimate may be made is to base it upon the cost of iron and manganese removal at Lowell, Massachusetts. The treatment plant[5] at Lowell, designed by Frank A. Barbour, Honorary Member and Past-President of This Association, cost approximately $20,000 per million gallons of daily capacity in 1914-15, probably equivalent to about $40,000 at the present time. Operating costs were $8.55 per million gallons in 1927, the latest year for which we have data. Analyses of the State Department of Public Health in that year show an average removal of iron of 3.2 p.p.m. but no figures are given for manganese removal. Assuming, however, that iron and manganese were present in equal amounts as early analyses show and taking annual depreciation and interest at 8 per cent the cost of iron and manganese removal was equivalent to $985, say $1000, per year per million gallons for each part per million of iron and manganese removed.

The cost of removal will not vary directly with the quantities of iron and manganese but we are attempting to arrive at some figure for economic loss, not at the cost of remedial action. We have assumed, therefore, that the Lowell remedial cost just balances the economic loss, that the figure given above represents this loss for each part per million of iron and manganese and that the loss will vary directly with the degree of impairment as we have assumed also in the case of other objectionable qualities.

The foregoing estimates of economic losses due to impaired water supply quality may be summarized in Table I.

It must be recognized that these figures are based upon very limited data and are not sufficiently accurate for general application. They do serve as a rough yardstick to illustrate the serious economic

[4]Jour. Am. W. W. Assoc. 25, 645 (May 1933).
[5]Jour A. W. W. A. 4, 129 (June 1917).

TABLE 1—ROUGH ESTIMATE OF ECONOMIC LOSS RESULTING FROM IMPAIRED QUALITY OF PUBLIC WATER SUPPLIES

TYPE OF IMPAIRMENT	ANNUAL LOSS PER MILLION GALLONS DAILY OF WATER CONSUMPTION
Sanitary Quality	$20,000 per death from water-borne typhoid
Color	$200 per p.p.m. over 10 p.p.m.
Turbidity	$400 per p.p.m. over 5 p.p.m.
Corrosiveness	Must be computed for each case
Hardness	$150 per p.p.m. over 100 p.p.m.
Iron and Manganese	$1,000 per p.p.m.

losses that result from the delivery of public water supplies of impaired quality.

It is probably true that the economic losses are not directly proportional to the degree of impairment, but, in view of the inadequacy of the available data, and the unavoidable reliance upon judgment to a considerable extent, it would be a matter of fictitious accuracy to attempt to use more involved assumptions.

The cost of removing impurities does not vary directly with the concentration of the impurities, except possibly as regards the expenditures for purification chemicals, and the economic loss, based upon purification costs, is not materially greater for treating a water with great impairment than for treating water with slight impairment. But, *it should be noted that the losses given in the table are those estimated to result from uncorrected impairments and not upon the cost of correcting these impairments.* Given a supply with specific qualities, this table permits a rough estimate as to whether purification is economically justifiable. If the total costs, fixed and operating, for purification are less than the economic losses due to the impairments, the purification of the supply is called for and a net economic saving is obtained.

ECONOMIC CONSIDERATIONS IN THE DESIGN OF WATER PURIFICATION PLANTS

Methods of Water Purification. The general methods of water purification have been outlined in a paper by one of the authors published in this JOURNAL.[6] They comprise storage, aëration, copper sulphate treatment of reservoirs, chlorination, filtration,—rapid and slow,—application of activated carbon, use of lime and other materials for corrosion control, water softening by lime-soda process or by zeolite filters and various combinations of the several methods. The following paragraphs deal with the economics involved in the selection of purification methods to be employed and in treatment plant design and operation.

[6]This Journal **48**, 451 (Dec. 1934).

According to one of the reports of the National Resources Board[7] there are in the United States about 7,100 public water supplies serving a population of 80 million with over 7,500 m.g.d. Data indicate that about 35 million people are supplied with filtered water, 20 million with water chlorinated only and 25 million with untreated water. Some 20 million people receive ground water and this group probably makes up the bulk of the population receiving untreated water. In this National Resources report it is estimated that the treatment plants in use represent an investment not far from $200,000,000[8] and that "an equal amount may eventually be required to furnish safe water to the entire country." This estimate of additional investment required is evidently predicated upon filtration of all surface water supplies.

While this goal is desirable, it is likely to be some time before it is reached. For example, the great water supplies of Boston, New York and Chicago,—although chlorinated,—are unfiltered. These supplies,—although reasonably safe from the public health standpoint,—have had at times qualities objectionable in the light of modern standards. Certainly in the case of Chicago and possibly in the case of New York and Boston it would not be difficult to justify filtration on broad economic grounds.

Storage. Long-time storage will reduce color and turbidity and will almost eliminate the danger of infection if short circuiting and current-conveyed pollution can be prevented. Six months actual storage should be sufficient to safeguard the sanitary quality of a water and would largely remove turbidity and reduce color.

Impounding reservoirs may cost from $100 to $1,000 per million gallons of capacity, the larger the reservoir in general the smaller the unit cost. Six months storage represents therefore an investment of from $18,000 to $180,000 per million gallons daily of consumption by means of which a reasonably safe, clear and moderately colored water may be obtained. Under some conditions, impairment due to tastes and odors or iron and manganese may offset in part improvement due to reduction in color and turbidity. From the standpoint of quality improvement alone, it is obvious that only in the larger supplies is long time storage economically justifiable if first cost alone is to be considered. On the other hand, annual maintenance and operation charges of storage reservoirs are relatively very low as compared with filtration.

Filtration. Although improvement in certain qualities may be obtained by one or more partial treatment methods it is generally

[7]Part III Report of the Water Planning Committee, November, 1934.

[8]Compare with $380,000 000 annual saving due to reduction in water borne typhoid resulting from existing purification plants.

true that only by filtration, combined with such other treatments as may be required to meet the particular conditions, can a surface supply of unsatisfactory characteristics be rendered wholly satisfactory. For example, chlorination can ordinarily be relied upon, if properly carried out, to remove the health hazard of a polluted supply, but chlorination cannot be expected to bring about any material improvement in color and turbidity; and in some cases it accentuates tastes and odors.

Copper sulphate treatment, aëration, activated carbon application and corrosion control by lime are usually, although not always, supplementary treatment processes. The adoption of any of these measures is usually determined by the public demand for good water rather than on the basis of economics. Nevertheless, economic factors enter into the problem. An illustration of this fact occurs in the matter of aëration, in conjunction with filtration, for the removal of tastes and odors. Adoption of aëration where low-lift pumping is involved means additional cost of pumping as the result of the head losses that occur. Under some conditions it would appear to be more economical in the long run to forego such aëration and rely on the use of activated carbon if the frequency of its use is not great.

Economic factors also enter into the choice of type of filters to be adopted, whether slow or rapid. Broadly speaking, the first cost of slow sand filters is greater than that of rapid filters per unit capacity, but the reverse is true as regards operation costs. Usually, considerations other than cost enter into the final decision. Sometimes, as at Greenfield, Mass.,[9] the advantage of both types of filters is obtained by combining coagulation by chemical treatment with filters of the slow sand type. By and large, however, modern practice in water filtration is to provide plants of the gravity rapid sand type. While industries continue to use the pressure type of rapid filter, seldom does a municipality install such filters at the present time. Pressure filters can produce satisfactory results under some conditions but to do so must be provided with adequate pre-treatment equipment similar to that required for plants of the gravity type, and when such is provided, little economy results from the use of pressure filters and some of their disadvantages remain.

Design. The success of a water treatment plant as regards results and low costs depends both upon design and operation. If the design is inadequate or poorly conceived, no amount of operating skill can achieve the best results. Similarly, a plant perfectly designed but operated carelessly or unintelligently will give unsatisfactory service. Defects in design, however, are often irrevocable

[9]Plant designed by Weston & Sampson, Consulting Engineers, Boston.

after the construction has been completed, whereas the type and quality of operation can usually be directed or changed at will. Design is, therefore, of great importance, and the features that are incorporated in the construction will influence for all time the adequacy and economy of operation. Decisions as to details of design will be governed in some cases by considerations of first cost and in other cases by questions of operating convenience and economy. Neither point of view can be lost sight of if a truly economical design is to be realized. This can be illustrated by reviewing the factors that govern the design of some of the principal features of a typical rapid sand filter plant, in sequence as the water passes through the plant.

Raw Water Intake. The type of intake will depend upon whether the source of raw water is a flowing stream or a relatively quiet lake or reservoir; whether the character of the water permits it to be drawn at a single point or requires selection from any one of several depths; whether the range between low and high water is extreme or minor; and whether the water normally contains much or little screenable solid matter. Most of these questions will have been settled during the preliminary investigations, and it will remain only to design structures suited to the topography and the foundation conditions, having in mind that water must be made available to the plant at all times in the manner most favorable for convenient operation.

Intake Screens. Screening is essential at intakes in practically all instances to exclude leaves and other coarse floating or suspended matter, and fish. The size of the treatment plant and the prevalence of débris in the source of raw water, will determine whether the screens should be hand-cleaned or equipped for automatic and mechanical cleaning. Hand cleaning is a tedious and laborious process at best and there is ever present the temptation of operators to neglect it. Except in very small plants and in those plants in which water is free from débris, other than seasonally as in the fall when leaves are falling, mechanical cleaning will usually be found desirable. In the larger plants, especially those taking raw water from rivers, which usually contain considerable débris at practically all times of the year, the expense of mechanical screens is justified, and they should be provided. Such screens may be operated at longer or shorter intervals during each day, depending upon the rate at which material is found to accumulate on them. The power cost for intermittent operation is insignificant, and the operating advantage is great by reason of the freedom from the necessity of hoisting and cleaning screens by hand. Mechanical screens must usually be protected against injury by means of coarse racks which will intercept large

objects; in some cases special precautions must be taken to prevent difficulties with ice.

Raw Water Pumping. Most filter plants are constructed as additions to existing supply systems. As a result it is commonly, although not always, necessary to introduce pumping to lift the untreated water sufficiently to provide the head necessary for its passage through the plant. The rate of pumping must be regulated to equal the rate of filtration. If the operator is to be spared the laborious work of continually adjusting valves and other devices to maintain this relation, automatic control must be provided. This is commonly accomplished by maintaining a substantially constant water level on the filters, by means of float-actuated throttling valves in the discharge pipe from the raw water pumps. Centrifugal pumps which are capable of throttling are most commonly used, being selected to deliver, unthrottled, rates slightly more than expected rates of filtration, so that regulation by a minimum of throttling can be effected.

Simplification of operation dictates that wherever possible the pumps should be self-primed. If the fluctuations of water level in the source of supply are not too great and if foundation and ground water conditions permit, this can be accomplished by locating the pumps below the lowest expected water level. If this is not practicable a reliable and completely automatic priming system should be designed so that any raw water pump will be immediately available for service without undue diversion of the operator's attention from his duties at the filter plant. Unless the raw water pumps are located in or very near the filter plant, remote control for starting and stopping the raw water pumps is essential for convenient operation.

Raw water pumps will usually be electric motor driven because of the conveniences of operation in the average sized plans. It is almost certain that electricity will be required for various purposes, but whether it will be purchased or generated will depend upon local conditions.

Application and Mixing of Chemicals. In most plants, it will be found convenient to apply chemicals with so-called dry-feed machines of which several serviceable types are available. This simplifies operation by eliminating the preparation of solutions. In very small plants, rates of dosing will be below the dependable range of dry feed machines, and solution feeding becomes necessary; the volumes of chemicals involved are small and the operation is, therefore, not unduly laborious. Also in very large plants, solution feeding may sometimes be found advantageous, especially when the volumes of chemicals required justify the installation of equipment to manufacture the required chemicals from cheaper raw materials.

To provide latitude in operation, chemical feed devices should be installed and equipped to permit the application of chemicals at various points in the process. For economy, the same machine is often arranged for use with different chemicals applied at different points as seasonal or other changes in the raw water may require. Machines for the application of chlorine or other disinfectants also should be so arranged as to permit of changing readily the application to any of several critical points.

The method of mixing will depend upon the nature of the treatment chosen for a particular raw water and is largely influenced by the personal preference and experience of the designer. Mixing is perhaps the topic upon which there is more difference of opinion than any other single phase of water purification. Mechanical mixing, with opportunity to vary the intensity and period of agitation, appears to give the operator the greatest latitude in adjusting the process to meet changing conditions in the water supply. Similarly, the time element relating to the formation of chemical floc and its optimum contact with the water under treatment can often best be regulated by adjustable gentle agitation just prior to sedimentation. Mechanical "flocculators" for this purpose will be justified in the interests of satisfactory operation when the nature of the water makes floc formation and control a critical factor.

Thorough mixing and flocculation are of distinct economic value. Proper preparation of the water to be settled and filtered often permits smaller sedimentation tanks than would otherwise be required and also results in the most efficient use of chemicals.

Sedimentation Tanks. If the continuous operation of a plant is not to be interrupted, sedimentation tanks must be in multiple units so that any single unit may be taken out of service for cleaning or for repair or adjustment of mechanisms without discontinuing the process of treatment. Proper arrangement and selection of type of gates and valves will contribute largely to the ease and convenience of operation and to the effectiveness of the plant. This, of course, also applies to gates and valves in all parts of the plant.

In cold climates, sedimentation tanks will usually be covered and if so, means of easy access and ventilation must be provided so that operators may conveniently enter for cleaning or maintenance. In large plants dealing with highly turbid waters or employing softening processes, mechanical equipment for the removal of sludge is justified.

Considerable care must be exercised in the design of inlets and outlets in order to utilize as advantageously as possible the entire capacity of the tanks. Economies in filter operation, particularly

Comp. by: PRB
Drawn by: JKB & JRW
Checked by: E.W.C.

GEOMETRIC MEAN OF B. COLI INDEX PER 100 c.c.

1 — 10 — 100 — 1000 — 10000

315

W-2a Above Jefferson Mfg. Co. at dam

W-2b Below Jeff. Mfg. Co. - Route 122 A

JEFFERSON BRANCH

W-5a — Babcock Brook - Route 62

W-5b — East Wachusett Brook - Bullard Rd.

EAST WACHUSETT BRK.

W-5 — Stillwater River - Old dam above West Sterling

W-1 — Main Stream at Mill St.

W-2 Jeff. Branch at Mill St.

W-5c — East Wachusett Brook at Route 64

W-3 Quinapoxet River - just below confluence of main stream and Jeff. Branch - Route 120

QUINAPOXET RIVER

STILLWATER

W-3a Chaffin Brook at Wachusett St. Unionville Pond outlet.

310

W-3b — Trout Brook at Manning St.

W-6 Stillwater River at Muddy Pond Road.

W-7 Waushacum Brook at Prescott St.

RIVER

W-4 Circular dam Shaft 1 Quabbin Aqueduct

W-4b — Wachusett Reservoir at Oakdale Bridge

W-4a Malden Brook at Thomas St.

W-8 Gates Brook at Temple St.

W-8b Malagasco Brook at Temple St.

W-8a Muddy Brook at Temple St.

W-4c Wachusett Reservoir at West Boylston Bridge

WA

COMMONWEALTH OF MASSACHUSETTS

RESULTS OF BACTERIOLOGICAL EXAMINATIONS

PREPARED UNDER THE DIRECTION OF THE JOINT BOARD
CONSISTING OF THE
METROPOLITAN DISTRICT WATER SUPPLY COMMISSION
AND THE DEPARTMENT OF PUBLIC HEALTH
Chapter 48-Resolves 1936

DECEMBER, 1937.

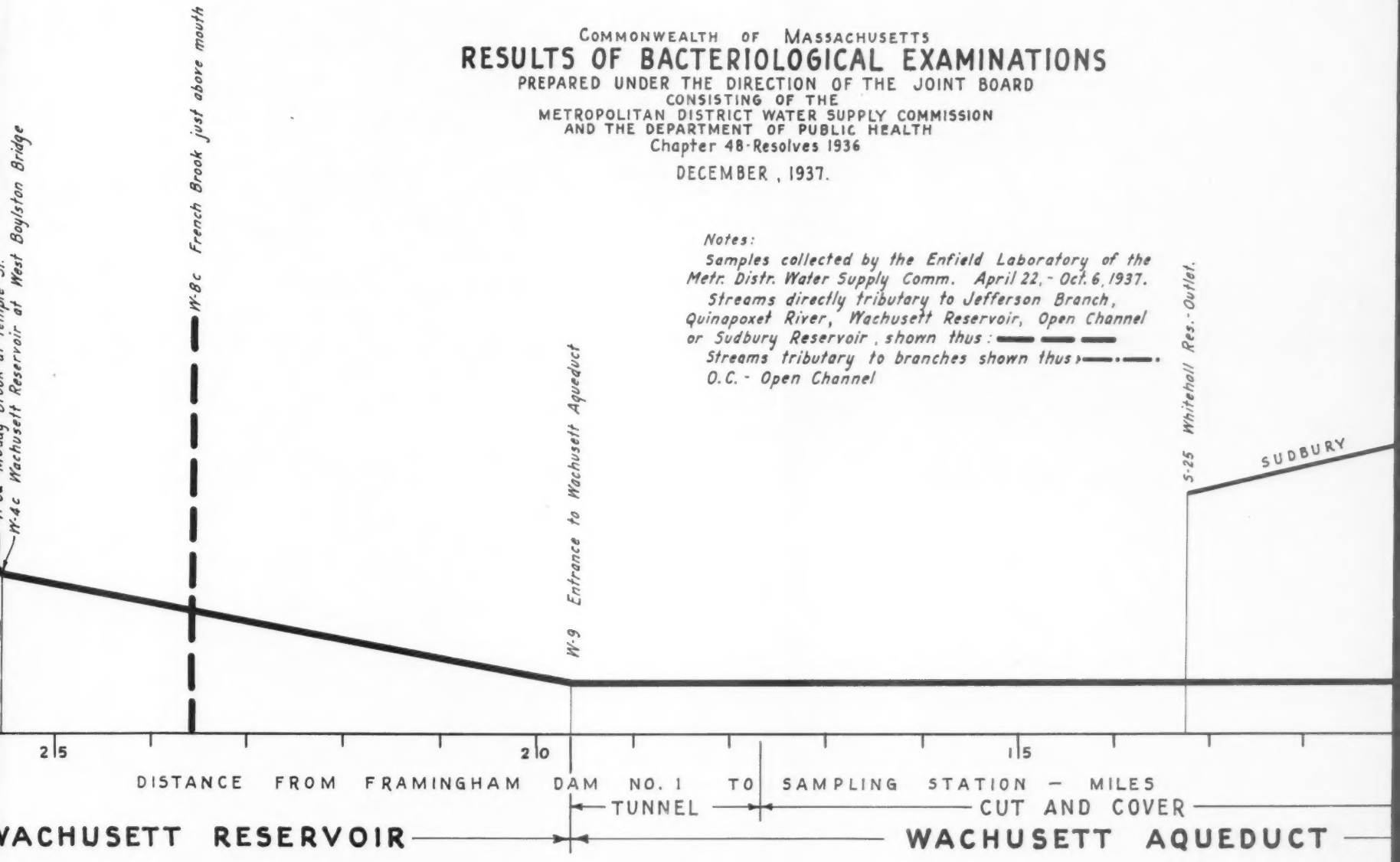
Notes:

Samples collected by the Enfield Laboratory of the Metr. Distr. Water Supply Comm. April 22, - Oct. 6, 1937.

Streams directly tributary to Jefferson Branch, Quinapoxet River, Wachusett Reservoir, Open Channel or Sudbury Reservoir, shown thus: ——— ———

Streams tributary to branches shown thus: ——·——·

O.C. - Open Channel



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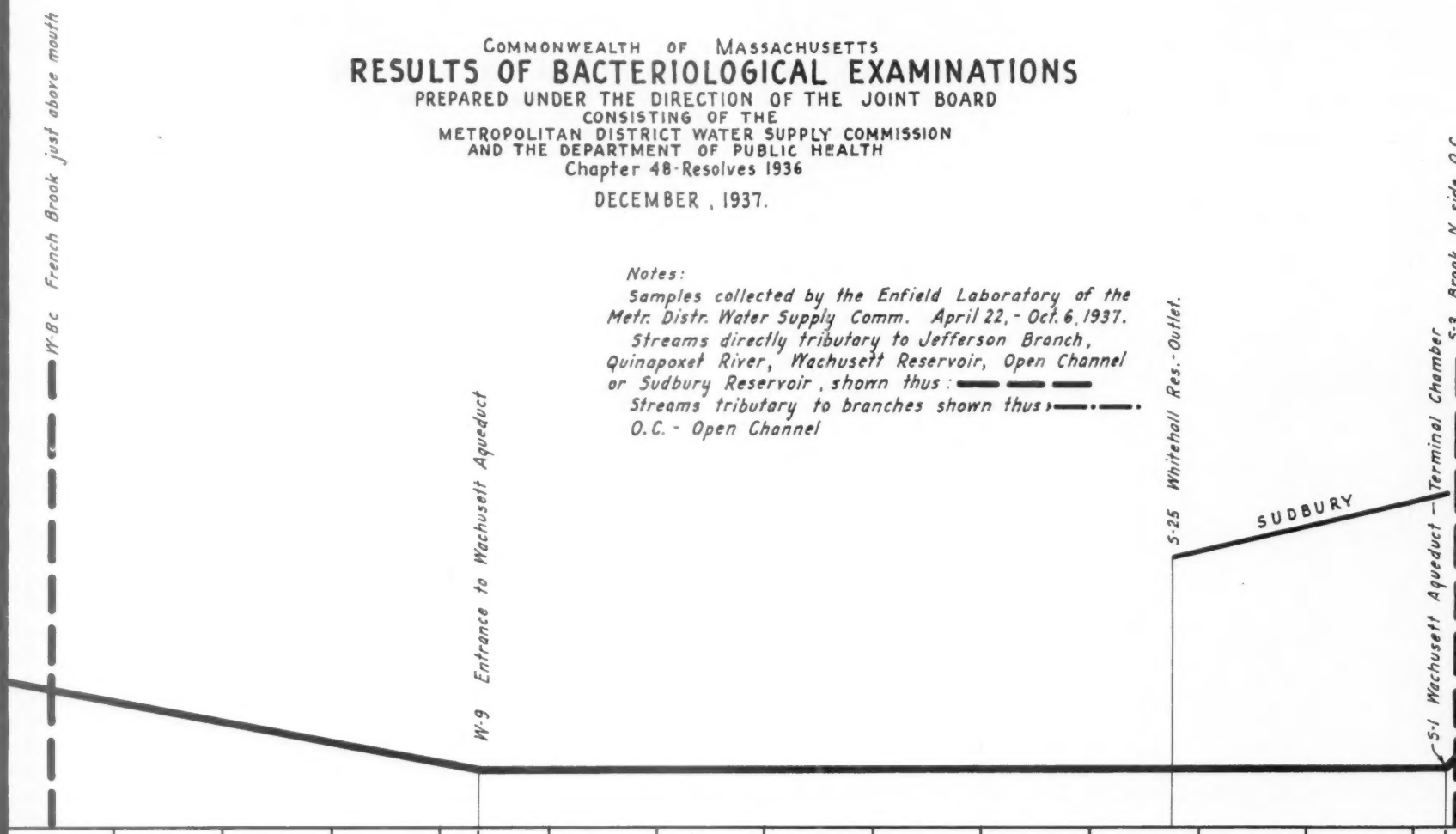
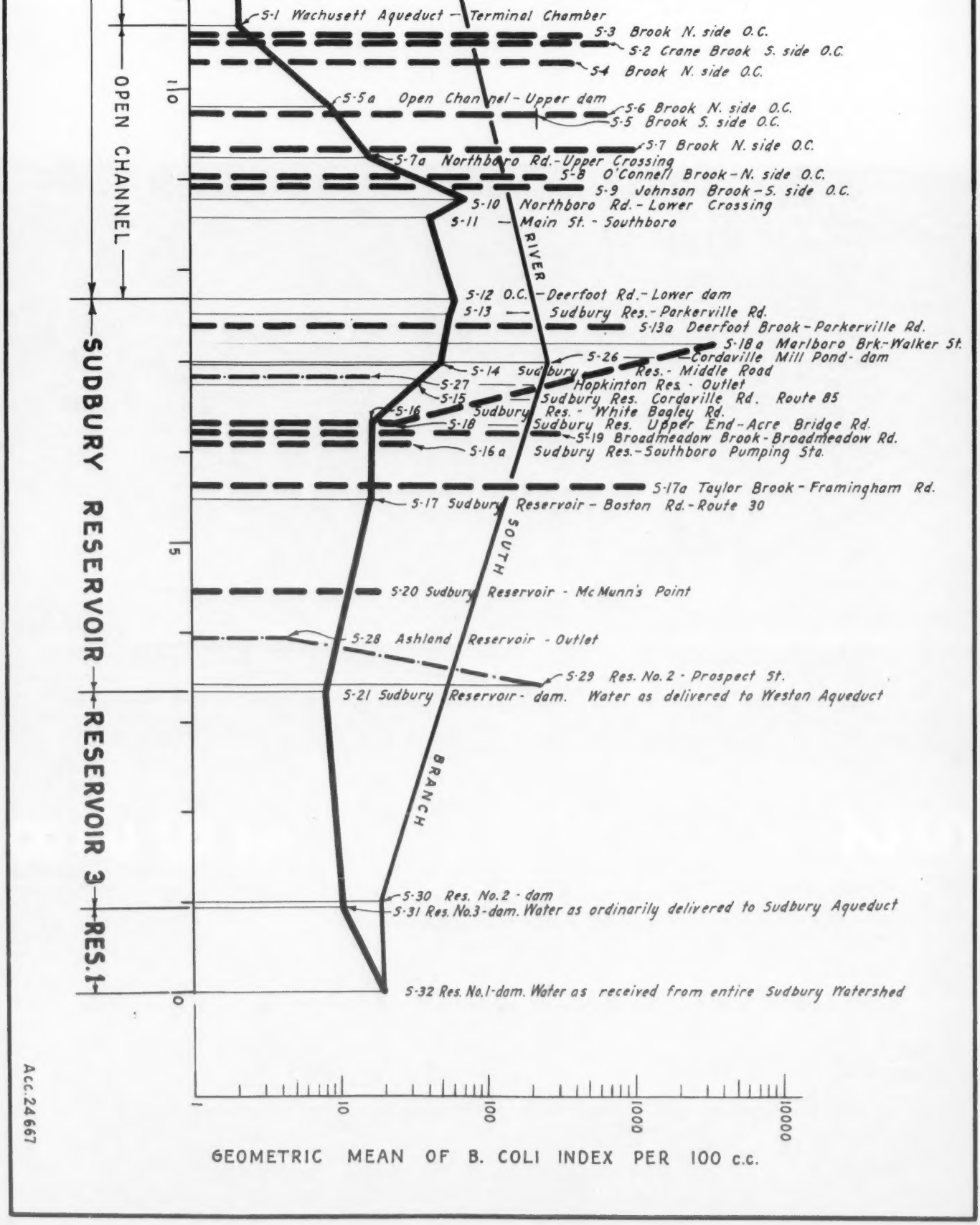
Streams tributary to branches shown thus: ———·———·

O.C. - Open Channel




S-1 Wachusett Aqueduct – Terminal Chamber
S-3 Brook N. side O.C.
S-2 Crane Brook S. side O.C.
S-4 Brook N. side O.C.
S-5a Open Channel – Upper dam
S-6 Brook N. side O.C.
S-5 Brook S. side O.C.
S-7 Brook N. side O.C.
S-7a Northboro Rd. – Upper Crossing
S-8 O'Connell Brook – N. side O.C.
S-9 Johnson Brook – S. side O.C.
S-10 Northboro Rd. – Lower Crossing
S-11 Main St. – Southboro
RIVER
S-12 O.C. – Deerfoot Rd. – Lower dam
S-13 Sudbury Res. – Parkerville Rd.
S-13a Deerfoot Brook – Parkerville Rd.
S-18a Marlboro Brk. – Walker St.
S-26 Cordaville Mill Pond – dam
S-14 Sudbury Res. – Middle Road
S-27 Hopkinton Res. – Outlet
S-15 Sudbury Res. Cordaville Rd. Route 85
S-16 Sudbury Res. – White Bagley Rd.
S-18 Sudbury Res. Upper End – Acre Bridge Rd.
S-19 Broadmeadow Brook – Broadmeadow Rd.
S-16a Sudbury Res. – Southboro Pumping Sta.
S-17a Taylor Brook – Framingham Rd.
S-17 Sudbury Reservoir – Boston Rd. – Route 30
SOUTH
S-20 Sudbury Reservoir – Mc Munn's Point
S-28 Ashland Reservoir – Outlet
S-29 Res. No. 2 – Prospect St.
S-21 Sudbury Reservoir – dam. Water as delivered to Weston Aqueduct
BRANCH
S-30 Res. No. 2 – dam
S-31 Res. No. 3 – dam. Water as ordinarily delivered to Sudbury Aqueduct
S-32 Res. No. 1 – dam. Water as received from entire Sudbury Watershed
OPEN CHANNEL
SUDBURY RESERVOIR
RESERVOIR 3
RES. 1
110
15
0
1
10
100
1000
10000
GEOMETRIC MEAN OF B. COLI INDEX PER 100 c.c.
Acc. 24667

as regards the length of filter runs and wash-water requirements depend to a marked degree upon the efficiency of the preparatory treatment. In this connection, the period allowed for sedimentation is an important factor, but the cost for sedimentation tank capacity must to some extent be balanced against filter operating costs.

Filters. Important features of design from the operating point of view are the control of rates of filtration through individual units, convenience of the numerous necessary valves and opportunity to vary and regulate washing rates.

Multiple filter units are necessary if filtration is not to be interrupted while a filter is being washed, and automatic rate controllers must be provided to distribute the load equally upon all units in service. In all but the smallest plants, the most frequently operated filter valves will be equipped for mechanical operation, usually by hydraulic cylinders, and the controls will be brought together on a panel or table opposite each filter unit. This convenience is sometimes omitted in the less elaborate industrial plants where ample labor is available for the rather arduous task of manual operation; even in such cases, valve stems should be extended to gate stands grouped on a common floor where the valves can be manipulated in an orderly sequence.

A wash-water rate-controller is desirable to limit the washing rate to that which the operator may select and to prevent injury to the filter by excessive rates. If auxiliary air or surface washing is to be employed, consideration must be given to providing convenient and close control if these auxiliary processes are to be effective.

Miscellaneous. Most plants include certain special features to deal with the peculiarities of the waters they are intended to treat, such as aëration, carbonation, and softening. The designer must give attention to providing all reasonable means for making these, as well as all other parts of the plant, flexible so that the operator will have the opportunity of varying the process to meet conditions not exactly anticipated when the basic requirements were determined.

In addition to planning operating conveniences, the designer's responsibility, of course, also extends to the structural and hydraulic details, which affect not only the life and adequacy of the finished works, but also operation, indirectly, in so far as structural design relates to water tightness and hydraulic design to a proper relation between the various structures and the ample capacity of connecting conduits. Economy of first cost must be considered but should not be permitted unduly to influence design to the sacrifice of adequacy and flexibility.

Many of the early gravity filters were enclosed in wooden tubs,

but most plants today have filter boxes of concrete. Recently, filter plants of steel construction have been built at costs that appear to be more advantageous than those for the conventional type. Steel has been used for many years in pressure filters, and it may be that under certain conditions the all-steel gravity filter plant will be advantageous. Like all innovations, however, time will be required to show what the weaknesses in this type of construction may be and also to provide opportunity for the development of measures to correct such weaknesses. It may well be that a steel plant with all the refinements, conveniences and long life of the conventional plant will be found to cost as much as the latter.

One feature of filtration plants that requires particular consideration from the standpoint of balancing costs against benefits is the clear-water basin. To provide storage for the purified water costs money, but on the other hand an inadequate reserve of filtered water to meet peak demands may be disastrous. No hard and fast rule can be laid down as to the proper ratio between filter capacity and size to clear-water basin. Each case must be considered by itself.

Softening. To soften or not to soften a particular supply is a question that can best be determined by whether the savings to the community in soap and gas consumption and in other ways are sufficient to offset the carrying charges for the softening plant. In New England, there are few public supplies of sufficient hardness to justify the expenditure of public funds for softening. In the hard-water regions, it is probably true that many more softening plants are needed than are now in use.

Hudson, in the article previously referred to, computed the net saving in cost of soap alone that would result from the softening of the water supplies of certain mid-west cities. For three cities, his figures indicated a net gain per capita averaging $1.25 annually. Hudson also presents the following equation from which may be calculated the hardness at which saving in soap will pay the cost of softening:

$$x=\frac{c-75d+75f}{f-d}$$

x=hardness of water supply in p.p.m., the softening of which will be paid for in soap economy alone.
75=demonstrated attainable hardness of water in p.p.m. from municipal softening plants.
f=soap waste per capita per year per p.p.m. hardness.
c=capital overhead charges, including superintendence, per capita per year.
d=cost of chemicals per capita per p.p.m. of hardness.

Another quick but somewhat less scientific method would be

to use the Dayton figure previously given for the economic loss due to hard water, multiplying this figure by the parts per million of hardness above 100 and by the number of million gallons daily of water consumption. The resulting figure will give the amount of money that may be spent annually for the carrying charges of a municipal softening plant that will turn out a water of 100 p.p.m. of hardness.

The selection of type of softening plant and the details of its design are matters that require careful study from the economic viewpoint. Whether to use zeolite softeners or lime-soda plants must be decided by local conditions and the character of the water to be softened. Zeolite softeners require a clear water such as well water or filtered water. The costs of chemicals in the two processes must be carefully determined and compared. Furthermore, the degree and skill of the attendance required must be considered and the relative expenditures for such attendance computed. Whatever type is finally selected, details of design must be carefully worked out to permit the efficient handling of the relatively large quantities of chemicals and the satisfactory disposal of the resultant sludge with the lime-soda process or the spent regenerating solution with the zeolite process.

Economic Considerations in the Operation of Water Purification Plants

While proper design of purification plants is essential, careful and efficient operation is equally essential. Poor operation of a good plant brings about low efficiency and high costs, whereas good operation of a poor plant can, to some extent, prevent such results.

Labor. Sometimes the control of labor costs is entirely out of the hands of those in responsible charge of the purification plant. In such cases the most efficient chief operator can do little, but in other cases where the responsibility is wholly his, there is opportunity for him to arrange his schedule of work in such a way as to prevent lost motion and waste of time, thereby keeping his force at a minimum consistent with proper operation.

In all but the smallest plants a technically trained superintendent or chief operator will more than save his salary by the efficiency and economy with which he will run the plant. Even in the small plants there should be part-time technical supervision.

Chemicals. Opportunity for waste or economy occurs to a marked extent in the chemical dosages employed. In large plants, this is a very important item, as a variation of a fraction of a grain per gallon in the alum dose, for example, means a considerable variation in the annual cost for this chemical. In small plants, the cost of chemicals does not become so great a proportion of total costs as in

large plants, but even in small plants, it is advisable to avoid waste of chemicals. This is particularly true of activated carbon, an expensive material.

A specific example of the saving in chemicals that can be accomplished by alert supervision may be cited. During August 1937, a slight taste and odor was present in the filtered water at Leominster, Massachusetts, although 67 lb. of activated carbon were being applied per million gallons of raw water. By changing the application of the carbon to the settled water just ahead of the filters, it was possible to remove completely the taste and odor with only 42 lb. of carbon, a saving in cost of $1.50 per million gallons. Many times, similar savings can be effected in the use of alum, lime, chlorine and other chemicals by a keen operator who is awake to his responsibilities.

Wash Water. Proper pre-treatment of the water by chemicals, coagulation and sedimentation permits longer filter runs and smaller wash-water consumption than where inefficient conditioning occurs. In case of reservoir waters, the judicious use of copper sulphate is a measure of economy at times when algæ growths are reducing the length of filter runs. Care in avoiding washing before it is really necessary and in keeping the time of washing as short as possible are also ways in which wash-water use may be kept low.

Maintenance. Expensive repairs may frequently be avoided by constant attention to routine maintenance. The use of the regular operating staff for maintenance work such as painting in the winter months and care of grounds in the summer tends to reduce the cost of maintenance and repairs. Here again high quality of operating supervision has a chance to demonstrate itself.

Data Relative to Cost of Construction and Operation of Water Purification Plants

Reliable and detailed estimates of cost of construction and of operation of purification plants are difficult to obtain; and when obtained, it is hard to make fair comparisons due to variation in local conditions. In some cases costs relate to gravity supplies and in others to supplies where pumping is involved. In recent years, plants have been built under PWA or WPA auspices; consequently their costs cannot fairly be compared with those of plants built under normal contractual conditions.

Construction Costs. Tables 2, 3 and 4 summarize the costs of construction of several plants of various capacities and of both the slow sand and rapid sand types and of water softening plants.

These tables show rather wide variations in cost of purification plants per million gallons daily capacity. In general, the larger the

TABLE 2—COST OF SLOW SAND FILTER PLANTS

PLACE	YEAR CONSTRUCTED	RATED CAPACITY M.G.D.	PRELIMINARY TREATMENT	DESIGN RATE OF FILTRATION M.G.D. PER ACRE	FILTERED WATER RESERVOIR CAPACITY M. G.	TOTAL COST ADJUSTED TO 1937 PER M.G.D. CAPACITY
Ashland, Wis.	1896	2	No	3	None	$ 65,900
Greenfield, Mass.	1935	2	Yes	5	0.55	84,500
Hartford, Conn.	1921 and 1927	25	No	4.6	15.	78,600
Keene, N. H. (Beech Hill)	1924-1925	0.75	No	5	1.5	119,500
Keene, N. H. (Stone Dam)	1924-1925	1.5	No	5	None	40,000
Philadelphia, Pa. (Torresdale)	1902-1904 and 1908	240.	Yes	4.9	50.	98,800
Philadelphia, Pa. (Upper Roxboro)	1903	16.8	No	3.0	8.	165,000
Springfield, Mass.	1910 and 1924	31.	Yes	4.8	17.	42,400
Wilmington, Del.	1910	15.	Yes	7.5	6.	78,600

TABLE 3—COST OF RAPID SAND FILTER PLANTS.

PLACE	YEAR CONSTRUCTED	DESIGN CAPACITY M.G.D.	DESIGN RATE OF FILTRATION M.G.D. PER ACRE	FILTERED WATER RESERVOIR CAPACITY M.G.	TOTAL COST ADJUSTED TO 1937 PER M.G.D. CAPACITY
Albany, N. Y.	1931	32	123	2.0	$44,200
Buffalo, N. Y.	1925	160	123	18.0	27,300
Burnt Mills, Md.	1934-1936	10	125	0.55	60,500
Cambridge, Mass.	1922 and 1932	24	136	4.0	54,300
Cleveland, O. (Division Ave.)	1916	150	124	20.0	42,700
Cleveland, O. (Baldwin)	1924-1925	165	124	135.8	60,400
Detroit, Mich. (Water Works Park)	1923	360	180	37.0	14,100
Detroit, Mich. (Springwells)	1929-1931	300	180	40.0	23,600
Greenwich, Conn.	1927-1928	4	125	1.0	92,000†
Salem-Beverly, Mass.	1935	8	119	4.0	101,000*

*Built under W. P. A. †Built under cost plus contract.

plant the smaller the unit cost. Furthermore, low-lift pumping stations add materially to the cost of construction and local conditions frequently result in special problems, the solution of which adds to the cost. The size of the clear-water basin in relation to the capacity of the plant is another factor that must be taken into consideration.

Provision for softening requires additional expenditure. The comparison of lime-soda plant costs with those of zeolite plants is hardly fair, because the lime-soda plants given in the table operate

TABLE 4—COST OF WATER SOFTENING PLANTS

PLACE	YEAR CONSTRUCTED	DESIGN CAPACITY M.G.D.	GENEBAL SOURCE OF SUPPLY	SOFTENING PROCESS	DESIGN RATE OF FILTRATION M.G.D. PER ACRE	SOFTENED WATER BASIN CAPACITY M.G.	TOTAL COST ADJUSTED TO 1937 PER M.G.D. CAPACITY
Bellevue, Pa.	1925	4	Wells	Zeolite	209	0.68	$24,500
Columbus, O.	1908-1923	54	Surface	Lime-Soda	120	16.50	28,300
Dallas, Texas	1929-1930	32	Surface	Lime	120	5.00	21,900
Mahoning Valley, O.	1932	40	Surface	Lime	125	1.00	84,000
Miami, Fla.	1924-1926	20	Wells	Lime	125	0.75	27,400
St. Louis, Mo. (Howard Bend)	1929	80	Surface	Lime	125	5.00	43,500

upon surface waters with other undesirable characteristics besides hardness. Figures for both types of plant are given, however, as a matter of information.

Operation Costs. Wide variations in operation costs also occur and for somewhat the same reasons as in the case of construction costs. Data upon operation costs are given in Tables 5 to 8.

TABLE 5—COST OF OPERATION OF SLOW SAND FILTERS

PLACE	YEAR	AVERAGE OUTPUT M G.D.	AVERAGE RATE OF FILTRATION M.G.D. PER ACRE	COST OF FILTRATION PER MILLION GALLONS	COST OF FILTRATION PER MILLION GALLONS PER PART PER MILLION OF REMOVAL OF: COLOR	TURBIDITY
Ashland, Wis.	1936	1.1	1.7	$5.33	...	
Greenfield, Mass.	1936	1.5	3.8	1.00	$0.10	$0.14
Hartford, Conn.	1935	18.2	3.4	2.51	0.28	
Philadelphia, Pa. (Torresdale)	1936	172.8	3.5	3.09	0.31	0 07
Philadelphia, Pa. (Upper Roxboro)	1936	15.4	2.8	4.65	0.66	0.08
Springfield, Mass.	1936	16.7	2.6	4.65	0.33	1.16
Wilmington, Del.	1935-1936	7.2	3.6	6.65	0.54	0.13

Generally, slow sand filters are used with comparatively clear water, although in certain cases they are used with turbid waters but after pre-treatment. It will also be found that in order to obtain effluents of low color when the raw water is high in color, it will be necessary to use either rapid sand filtration or some form of pre-treatment with slow sand filters.

The figures showing costs of color, turbidity and hardness removal per part per million are interesting for comparison with the estimated economic loss resulting from these qualities. This comparison is given in Table 8.

From this table, it will be seen that for the low figures given in

TABLE 6—COST OF OPERATION OF RAPID SAND FILTERS

PLACE	YEAR	AVERAGE OUTPUT M.G.D.	AVERAGE RATE OF FILTRATION M.G.D. PER ACRE	COST OF FILTRATION PER MILLION GALLONS	COST OF FILTRATION PER MILLION GALLONS PER PART PER MILLION OF REMOVAL OF: COLOR	TURBIDITY
Albany, N. Y.	1934-1935	20.3	78.0	$14.21	$1.02	$2.37
Buffalo, N. Y.	1936	125.4	98.2	3.00*	0.75	0.07
Burnt Mills, Md.	1936	4.6	58.0	..	....	....
Cambridge, Mass.	1936	13.8	78.0	8.77*	0.37	4.38
Cleveland, O. (Division Ave.)	1936	67.0	55.4	7.05*	....	0.54
Cleveland. O. (Baldwin)	1936	95.8	72.0	4.75	2.38	0.53
Detroit, Mich (Water Works Park)	1936-1937	153.3	76.6	3.24*	....	0.15
Detroit, Mich. (Springwells)	1936-1937	113.6	68.2	2.79*	....	0.11
Greenwich, Conn.	1928-1931	5.2	118.0	16.78*	0.76	5.59
Salem-Beverly, Mass.	1936	6.0	89.4	15.04*	0.44	....

*Includes low lift pumping

TABLE 7—COST OF OPERATION OF WATER SOFTENING PLANTS

PLACE	YEAR	AVERAGE OUTPUT M.G.D.	AVERAGE RATE OF FILTRATION M.G.D. PER ACRE	COST OF SOFTENING PER MILLION GALLONS	COST OF SOFTENING PER MILLION GALLONS PER PART PER MILLION OF HARDNESS REMOVED
Bellevue, Pa.	1927	3.5	183	$19.24*	$0.11
Columbus, O.	1936	35.0	78	14.28**	0.08
Dallas, Texas	1935-1936	24.8	93	8.21†	0.09
Mahoning Valley, O.	1936	14.3	45	14.35††	0.33
Miami, Florida	1936-1937	16.4	102	27.00‡	0.15
St. Louis, Mo. (Howard Bend)	1935-1936	47.2	74	9.80††	0.12

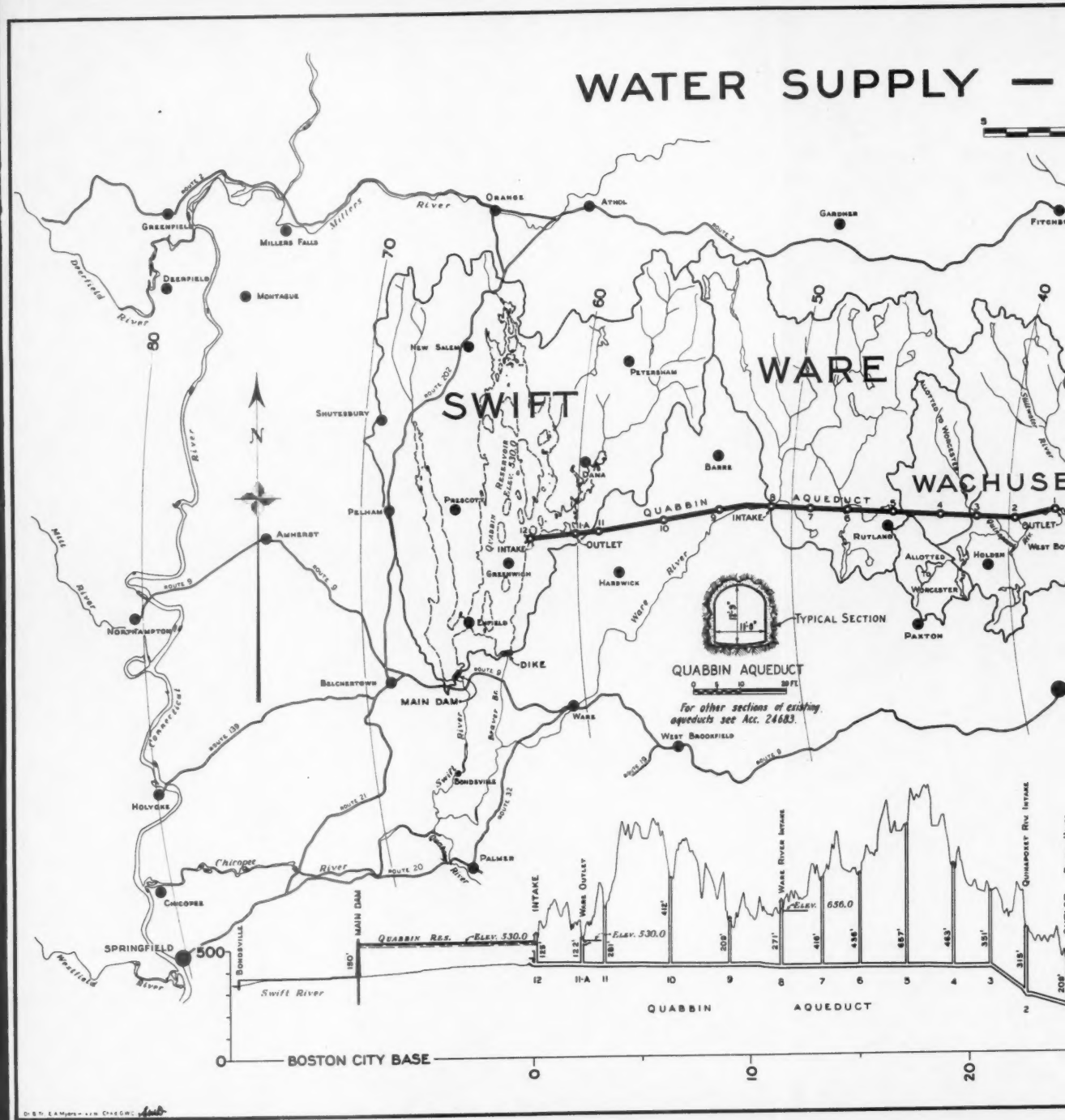
*Includes increase in pumping due to softening plant.
**Includes low lift pumping.
†Does not include power.
††Does not include low lift pumping.
‡Includes high and low lift pumping.

TABLE 8—COMPARISON OF ESTIMATED ECONOMIC LOSS DUE TO COLOR, TURBIDITY AND HARDNESS WITH THE COST OF PURIFICATION PLANT OPERATION.
(Total Operation Cost charged against the removal of each constituent in turn)

	ESTIMATED ECONOMIC LOSS	YEARLY COST PER PART PER MILLION PER MILLION GALLONS DAILY (EXCLUSIVE OF FIXED CHARGES) REMOVAL BY SLOW SAND FILTERS	REMOVAL BY RAPID SAND FILTERS	REMOVAL BY SOFTENING PLANTS
Color	$200	$36.50 to 240.00	$135.00 to 870.00	....
Turbidity	400	2.56 to 423.00	2.56 to 2040.00	....
Hardness	150	.....	.....	$2.92 to 120

the ranges in cost the *total* operating costs are less than the economic loss due to the constituents removed. On the other hand where the raw water is low in a particular constituent the cost per unit is, of course, abnormally high. The correct procedure would be to compare the total benefits derived from purification with the total annual charges, operation and fixed; therefore, Table 8 is principally of academic interest. Furthermore, as previously pointed out, local conditions in each case materially influence the costs and *comparisons between plants must be made with caution and only after full knowledge of details influencing design, construction and operation of the particular plants*. In some cases the costs include pumping; in other cases they do not.

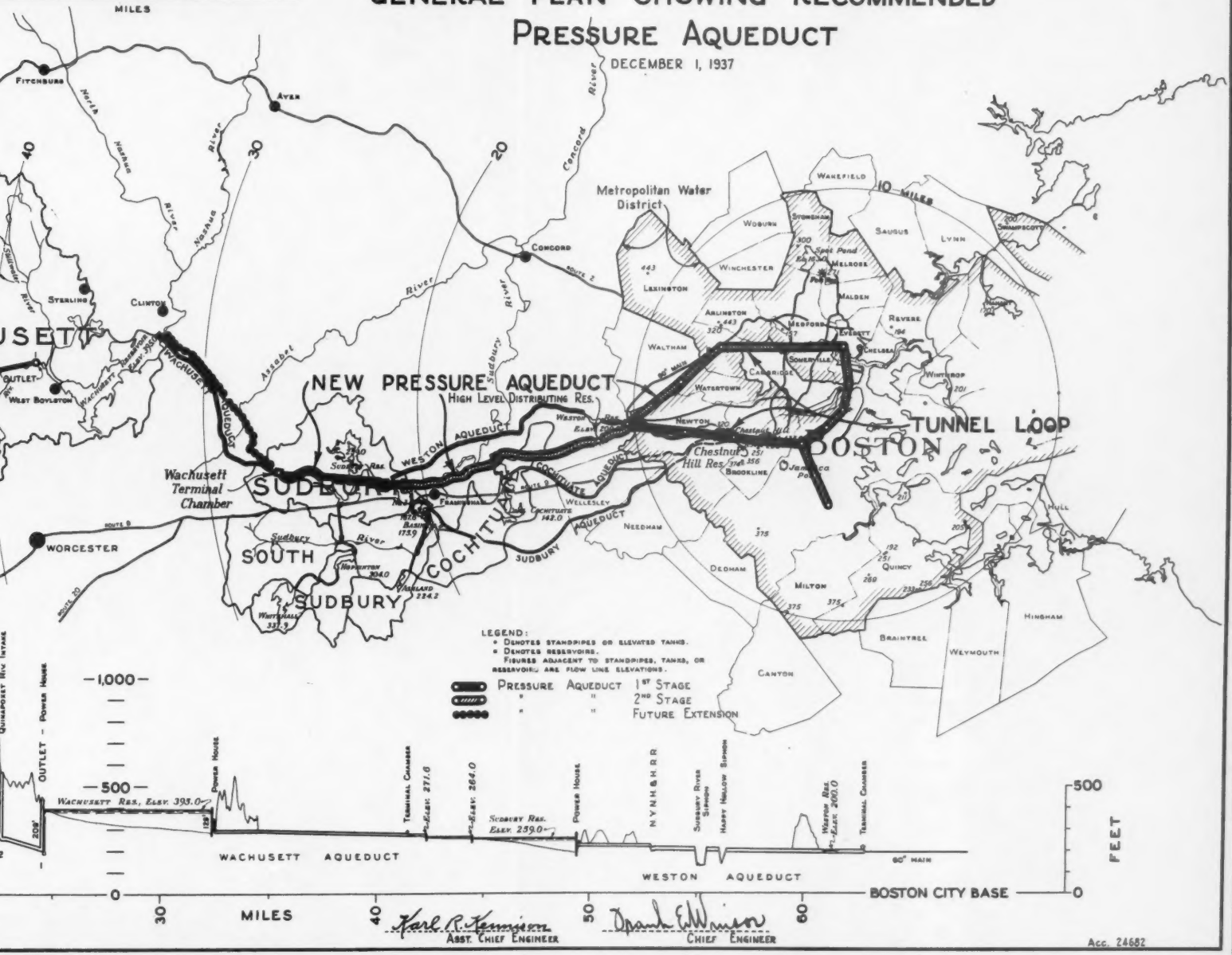
Acknowledgment. The authors wish to express their gratitude to the many water works officials who have furnished data and to Mr. George C. Houser, Research Engineer of Metcalf & Eddy, for his patient and laborious compiling of the data obtained.


WATER SUPPLY —
SWIFT
WARE
WACHUSE
N
GREENFIELD
MILLERS FALLS
Millers River
ORANGE
ATHOL
GARDNER
DEERFIELD
Deerfield River
MONTAGUE
NEW SALEM
SHUTESBURY
PETERSHAM
BARRE
PRESCOTT
PELHAM
AMHERST
Mill River
NORTHAMPTON
Connecticut River
BELCHERTOWN
GREENWICH
INTAKE
OUTLET
QUABBIN
AQUEDUCT
INTAKE
OUTLET
RUTLAND
HARDWICK
Ware River
ENFIELD
DIKE
MAIN DAM
WARE
WEST BROOKFIELD
HOLYOKE
BONDSVILLE
Swift River
Chicopee River
CHICOPEE
SPRINGFIELD
PALMER
Westfield River
ALLOTTED
WORCESTER
HOLDEN
PAXTON
WEST BO
TYPICAL SECTION
QUABBIN AQUEDUCT
For other sections of existing aqueducts see Acc. 24683.
MAIN DAM
QUABBIN RES. ELEV. 530.0
INTAKE
WARE OUTLET
ELEV. 530.0
WARE RIVER INTAKE
ELEV. 656.0
QUINAPOXET RIV. INTAKE
SWIFT RIVER
BOSTON CITY BASE
QUABBIN
AQUEDUCT

METROPOLITAN BOSTON

GENERAL PLAN SHOWING RECOMMENDED PRESSURE AQUEDUCT

DECEMBER 1, 1937



THE NEED FOR IMPROVEMENTS IN THE DISTRIBUTION SYSTEM OF THE METROPOLITAN WATER DISTRICT

BY EUGENE C. HULTMAN*

[*Read February 17, 1938*]

As soon as the Metropolitan Water District completes its present program of new construction, it will be adequately provided for so far as quantity at the source is concerned.

Development of Water Supply Sources. The development of the sources may be described very briefly, as a continual extension westerly in advance of the steady encroachment of population and its attendant pollution on the older watersheds. See Plate I.

Boston's first real water supply worthy of the name, from Lake Cochituate in 1848, proved to be adequate for a period of about 25 years. An additional supply, the Sudbury System, was adequate for another 25-year period. This supply was first obtained in 1872 by a diversion from the Sudbury River into Lake Cochituate, and the finished system, including the Sudbury Reservoir and Aqueduct, was put into operation in 1878. A further addition to the supply, the Wachusett System, has served its purpose admirably for another 30- or 35-year period. This supply was first obtained from the Nashua River in Clinton in 1898 by the then newly organized Metropolitan Water District, and the Wachusett Reservoir was filled for the first time in 1908. Looking ahead into the future, a comparison of the safe yield with the estimated requirements indicates that the water supply needs of the District will upon completion of the present program of extension to the Ware and Swift supplies be well taken care of for approximately another 40 years.

This last extension has been under construction since 1926. First the Quabbin Aqueduct Tunnel, 11 ft. wide and 12 ft. 9 in. high in solid rock was completed as far as the Ware River in 1931 and extended to its full length of 24.6 miles to the Swift River in 1935. See Plate I.

The Ware River Intake Works have been in operation since 1931, dropping the flood flows 260 ft. into the tunnel below, 1½ times the drop over Niagara Falls, but it makes at present only a small contribution to the water supply on account of inadequate storage capacity in Wachusett Reservoir into which the diversion is made. The works are designed so that water from the Ware River

*Chairman, Metropolitan District Water Supply Commission, Boston, Mass.

FIG. 1. QUABBIN DIKE, LOOKING WEST

FIG. 2. QUABBIN DAM UNDER CONSTRUCTION

can be diverted to and stored in either Wachusett Reservoir or Quabbin Reservoir on the Swift River, but the full benefit of the Ware works, like that of the Swift, must wait for the completion of Quabbin Reservoir.

The smaller of the two large earth dams necessary to impound the water in this reservoir is now completed. (Fig. 1.) This embankment of 2,500,000 cu. yd. is 2,140 ft. long across the top, 135 ft. high above the original brook bed and 264 ft. above the foundation in sound ledge.

The Main Dam is under construction by the method of hydraulic fill and should be completed next year. It will contain 4,000,000 cu. yd., and will be 2,640 ft. across the top, 170 ft. high above the river bed and 295 ft. above the foundation in sound ledge (Fig. 2).

Yield of Sources. Until Quabbin Reservoir is completed, Wachusett Reservoir continues to be the chief source of supply, and this has to be supplemented by Sudbury Reservoir and other sources. A tabulation of the safe yield of the District's sources shows the extent to which we are now forced to rely upon the older, undesirable and now polluted sources, a condition which requires immediate remedial action. See Table 1.

TABLE 1—SAFE YIELD OF METROPOLITAN WATER SOURCES

	AVAILABLE DRAINAGE AREA SQ. MILES	CONTINUOUS SAFE YIELD (EACH AREA OPERATED NOT BY ITSELF BUT AS A PART OF THE WHOLE) M.G.D.	STORAGE SAFELY AVAILABLE BIL. GAL.	ADDITIONAL STORAGE RESERVED TO PROTECT QUALITY OF SUPPLY BIL. GAL.
Before Completion of Quabbin Reservoir				
Wachusett	107.69[a]	85[b]	45	20
North Sudbury	27.68	24	7	2
Ware	96.8	11	0	0
South Sudbury	46.98	19	3	2
Cochituate	17.4	8	2	3
		147[b]		
After Completion of Quabbin Reservoir				
Wachusett	107.69[a]	99[b]	45	20
North Sudbury	27.68	24	7	2
Ware	96.8	40	0	0
Swift	185.9[c]	154	395	20
South Sudbury	10.61[d]	3	2	1
		320[b]		

[a]After deducting 1.15 square miles tributary to East Waushacum Pond, diverted July 1, 1937; and including the entire area of 20.7 square miles from which Worcester in 1926 and 1931, was granted the right to divert at a yearly average rate of 10 mil. gals. daily.

[b]After deducting 10 mil. gals. daily which Worcester has the right to divert from an area of 20.7 square miles.

[c]Not including 1.8 square miles, of which 1.0 has been diverted into and the remaining 0.8 is naturally tributary to North Pond in the town of Orange, from which occasional floods will flow into Quabbin Reservoir.

[d]Including Whitehall and Hopkington reservoirs only.

Lake Cochituate and the Cochituate Aqueduct from it to Chestnut Hill Reservoir still have to be used in emergencies on account of lack of carrying capacity in the aqueducts from Sudbury Reservoir. Hence it is retained in use, in spite of the fact that Natick, Framingham and Wayland have spread their steady growth over its shores and watershed and in the face of continued and increasing demand on the Legislature that this lake be devoted to recreational use. This supply was used regularly in considerable volume through 1911 and to a lesser extent through 1920. The only reason we do not find it necessary to use it this year is because this, and also fortunately the last few years as well, have been wet years. Whenever we have two dry years in succession, we have to use it. For example, in 1931, more water was used from Lake Cochituate than in any year since 1905. Ninety years ago when this lake was developed, no land was purchased to protect the watershed, and the District now owns in places only a few feet from the shore.

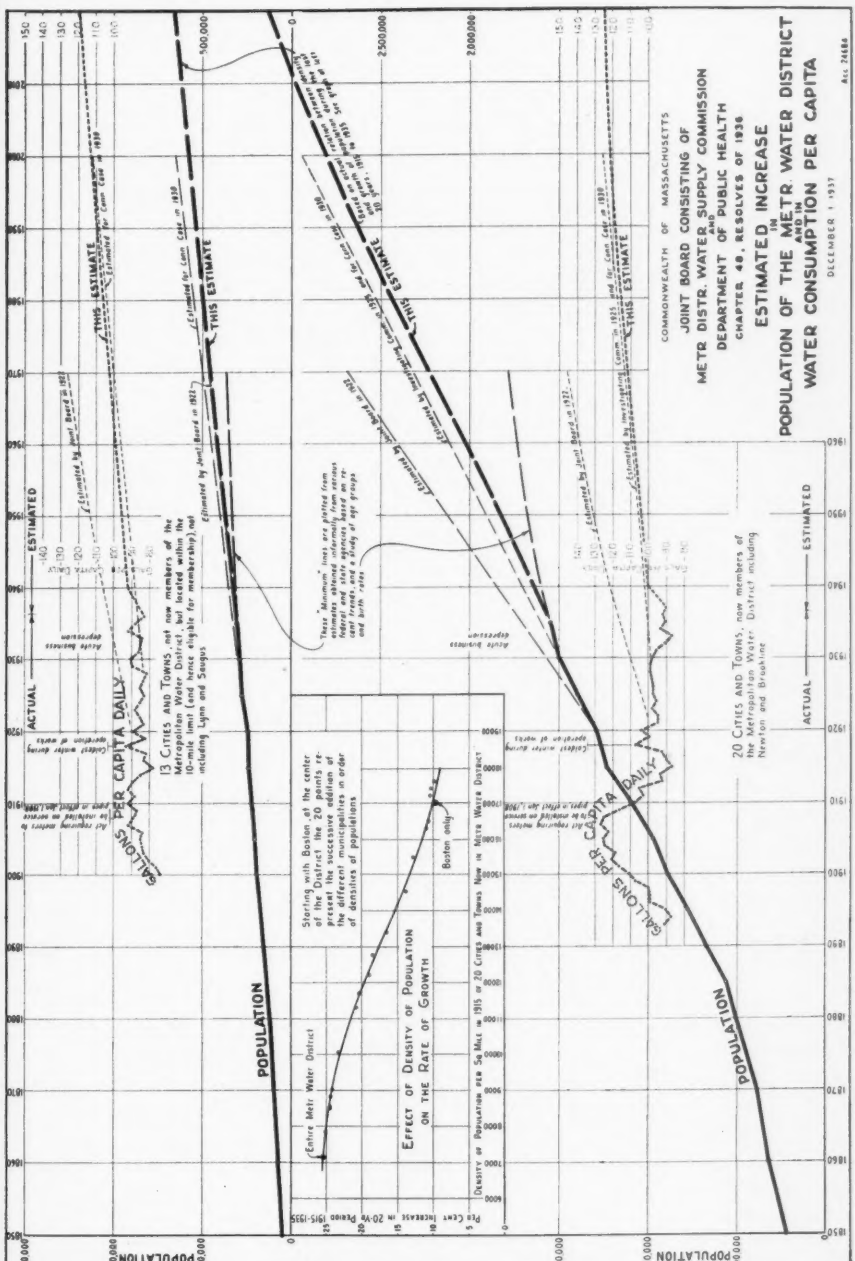
As for the South Sudbury source, there has been such an increase in pollution from the population and industries in Ashland, Westborough and Hopkinton as to force practical discontinuance of this source after 1912. However, the actual water shortage was so great prior to the business depression that the District spent about $640,000 in 1927 to divert water from the less polluted headwaters in Ashland, Hopkinton and Whitehall reservoirs as an emergency measure.

Even the North Sudbury area, which continues in daily use, is subject to pollution on account of the population and industries in Marlborough and Southborough. Nevertheless, the present aqueduct system requires the entire supply from Wachusett Reservoir to be passed through the Sudbury Reservoir en route to the District.

Future Requirements. In addition to the fact that fortunately we are not now experiencing a period of drought, another reason that has enabled us to use water from only our best sources during the last few years is that the per capita consumption in the District has been reduced about 10% during the business depression,—but it is now showing an increase. In order to make safe provisions for the future, we cannot do less than assume that the rate of consumption will at an early date return to normal. See Table 2. It would not be wise to provide for a rate of population growth less than that which has been observed in the District during the last 20 years, which rate of growth was decidedly less than the earlier pre-war rate (Fig. 3). The District is in the unique position of having a large number of separate municipal units, on which the census gives us data, varying from the densely populated center to the more sparsely populated margins, and the small insert in Figure 3 shows a remark-

TABLE 2—GROWTH OF POPULATION AND WATER CONSUMPTION IN THE METROPOLITAN WATER DISTRICT, IN MILLION GALLONS DAILY

YEAR	20 CITIES AND TOWNS NOW MEMBERS OF METROPOLITAN WATER DISTRICT					M.G.D. SUPPLIED TO NEARBY CITIES AND TOWNS WHICH MAY JOIN THE DISTRICT FROM TIME TO TIME	TOTAL M.G.D. TO BE DISTRIBUTED WITHIN THE 10-MILE LIMIT. (ADD 20% TO THESE YEARLY AVERAGES TO OBTAIN PEAK OF ONE-WEEK DURATION)	M.G.D. SUPPLIED TO NON-MEMBERS SUCH AS FRAMINGHAM, NATICK, HOSPITALS, ETC.	M.G.D. SUPPLIED TO WORCESTER BY PUMPING FROM WACHUSETT AREA	TOTAL DEMAND ON DISTRICT SOURCES M.G.D.
	POPULATION	APPROX. DENSITY PER SQ. MILE	PER CAPITA GAL. DAILY (AND % OF SERVICES METERED)	TOTAL M.G.D. CONSUMED	CREDIT M.G.D. FROM LOCAL SOURCES NEWTON AND BROOKLINE					
1850	226,510	1,310	..	..	..	..	..	..	..	..
1860	321,189	1,850	..	..	..	..	..	..	..	..
1870	389,625	2,250	..	..	..	..	..	..	..	..
1880	508,884	2,940	..	..	..	..	..	..		..
1885	565,521	3,260	..	..	..	..	..	..		..
1890	674,535	3,890	..	..	..	..	..	..		..
1895	779,581	4,500	90.6	70.82	..	..		..		
1900	897,475	5 180	111.4 (8)	100.00	..	..		..		
1905	974,877	5,620	125.3 (11)	122.13	..	..		..		
1910	1,098,048	6,340	107.6 (38)	118.22	—6 13	..	112.09	0.87	..	112.96
1915	1,234,790	7,130	87.1 (67)	107.52	—5.58	..	101.94	1.06	0.13	103.13
1920	1,290,651	7,450	104.1 (74)	134.40	—7.13	..	127.27	1.38	..	128.65
1925	1,398,702	8,080	97.6 (96)	136.52	—7.66	..	128.86	3.06	2.58	134.50
1930	1,498,253	8,650	97.5 (99)	146.11	—8.95		137.16	3.56	2.39	143.11
1935	1,548,282	8,940	90.9 (100)	140.71	—9.45		131.26	3.89	..	135.15
1936			92.1 (100)	143.48	—9.62		133.86	3.86	..	137.72
ESTIMATED										
1940	1,635,000	9,430	100	163.5	—10	0	153.5	4		157.5
1950	1,825,000	10,530	104	190	—10	12.6	192.6	5	5	202.6
1960	2,025,000	11,660	108	219	—5	31.5	245.5	6	10	261.5
1970	2,230,000	12,880	112	250	..	42.6	293	7	10	310
1980	2,435,000	14,050	116	282	..	48.3	331	8	10	349
1990	2,630,000	15,200	119	313	..	55.1	368	9	10	387
2000	2,810,000	16,200	122	343	..	61.6	405	10	10	425
2010	2,980,000	17,200	124	369	..	68.2	437	11	10	458
2020	3,130,000	18,100	125	391	..	74.7	466	12	10	488
2030	3,270,000	18,900	125	408	..	80.7	489	13	10	512

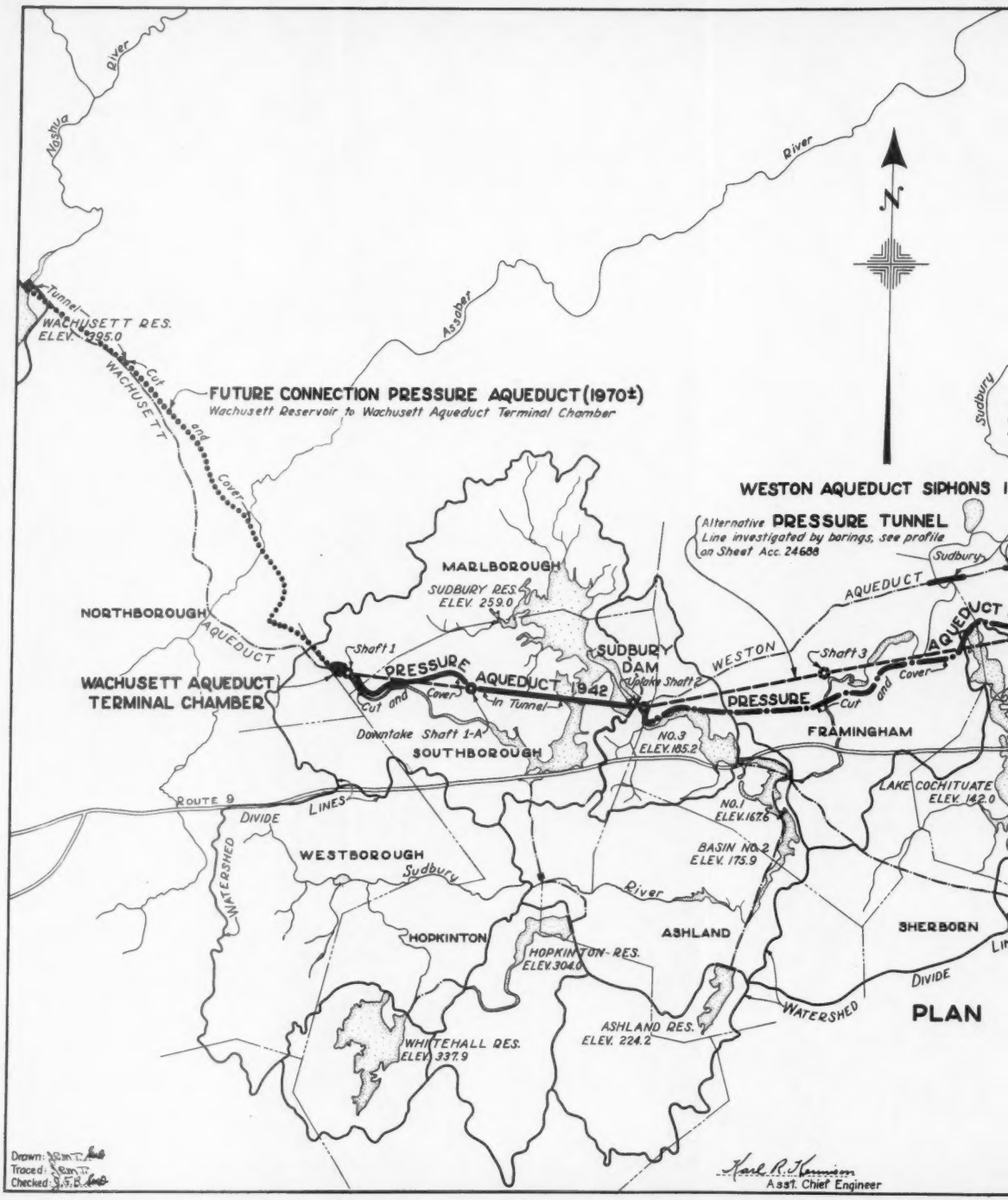

COMMONWEALTH OF MASSACHUSETTS
JOINT BOARD CONSISTING OF
METR DISTR. WATER SUPPLY COMMISSION
AND
DEPARTMENT OF PUBLIC HEALTH
CHAPTER 40, RESOLVES OF 1936
ESTIMATED INCREASE
IN
POPULATION OF THE METR. WATER DISTRICT
AND IN
WATER CONSUMPTION PER CAPITA
DECEMBER 1, 1937
POPULATION
GALLONS PER CAPITA DAILY
ACTUAL
ESTIMATED
13 CITIES AND TOWNS, not now members of the Metropolitan Water District but located within the Metropolitan Water District (as bounded by Chapter 92, Acts of 1895), now supplied by Metr. Dist. including Lynn and Saugus
20 CITIES AND TOWNS, now members of the Metropolitan Water District including Newton and Brookline
THIS ESTIMATE
EFFECT OF DENSITY OF POPULATION ON THE RATE OF GROWTH
Entire Metr. Water District
Boston only
Per Cent Increase in 20-yr Period 1915-1935
Density of Population per Sq. Mile in 1915 or 20 Cities and Towns Now in Metr. Water District
Starting with Boston, at the center of the District, the 20 points represent the 1915 population of the different municipalities in order of densities of populations
These "Maximum" lines are plotted from estimates obtained informally from extrapolation of past trends, from current trends, and a study of age groups and birth rates
Estimated by Joint Board in 1932
Estimated by Comm. Corp. in 1930
Estimated by Investigating Comm. in 1935 and Joint Board in 1936
Estimated by Joint Board in 1936
Estimated by Comm. Corp. in 1930
Estimated by Joint Board in 1932
Estimated by Investigating Comm. in 1935 and

ably definite relationship between density of population and rate of growth. The estimate for the future growth of population simply uses the past growth of the dense center of the District, for example, Boston, Somerville, Chelsea, etc., as a criterion and assumes that the growth of the large district will be curtailed to the same extent when they reach the same density. The result indicates that the population of the 20 cities and towns now members of the water district will be 1,825,000 in 1950.

It is estimated that the total water supply demand on District sources which averaged about 135 m.g.d. in 1937, will be somewhat more than 200 m.g.d. in 1950 (Fig. 3). Even if we take the estimates of others, who have on the basis of age groups and birth rates predicted a smaller rate of growth of population, the consumption will probably exceed 185 m.g.d. in 1950.

Nearby municipalities not in the Water District will be forced to come in due to inability to protect or increase their own local supplies. About two-thirds of the yield of these local supplies, a total of about 30 m.g.d. if we add the amounts diverted by Newton and Brookline, already members of the District, is taken in one way or another, by means of wells or surface reservoirs, from the watershed of the Charles River. This is a diversion from the river which ultimately is carried away by the Metropolitan sewers, so that a great part of the clean upland flow is taken from the Charles River in the dry season of the year when it and the Charles River Basin are mostly used for boating, bathing, etc. This flow is greatly needed to preserve the quality of the water in this great park in suitable condition for recreational uses as well as for the comfort of the many whose homes are adjacent to it. Not only should further abstractions from the Charles River drainage area be prohibited, but the time may not be far distant when steps to secure the return of some of the present diversions may be essential.

As the consumption increases, the difficulty of maintaining adequate delivery to the cities and towns in the District through the present system of aqueducts and pipe lines continually increases. Also the problem of pollution on the watersheds is already a major one and rapidly becoming more serious. The Legislature has recognized these facts and by Chapter 48 of the Resolves of 1936 authorized and directed the Metropolitan District Water Supply Commission and the Department of Public Health, acting jointly, to investigate methods of improving the distribution of water from the sources of supply to the Metropolitan Water District, more adequately preventing pollution of the sources of water supply, and protecting the purity of the water. The report of this Joint Board is now before the Legislature.



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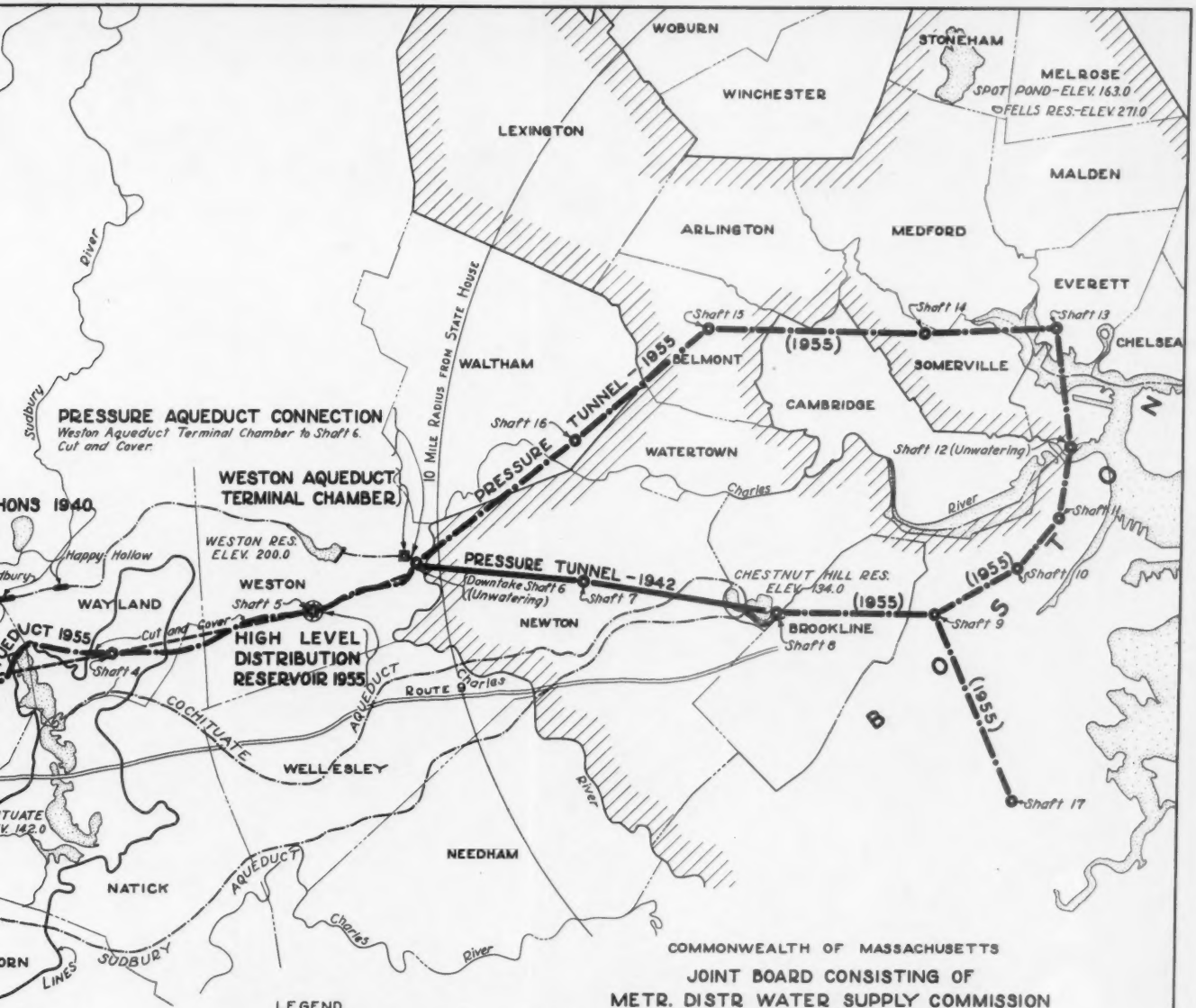
Karl R. Harrison
Asst. Chief Engineer



LEGEND

- 1st Stage – Recommended for immediate construction (1938-1942)
- 2nd Stage – Estimated to be needed about 1955.
- 3rd Stage – Estimated to be needed about 1970.
- Alternative tunnel line studied.
- Limits of Existing Metropolitan Water District.

COMMONWEALTH OF MASSACHUSETTS

JOINT BOARD CONSISTING OF

METR. DISTR WATER SUPPLY COMMISSION

AND

DEPARTMENT OF PUBLIC HEALTH

CHAPTER 48 - RESOLVES OF 1936

NEW PRESSURE AQUEDUCT FOR METROPOLITAN SUPPLY

0 1 2 MILES

DECEMBER 1, 1937

Chief Engineer

Acc. 24687

Sanitary Survey of Watersheds

The first recommendation of the Board is for the immediate undertaking of a program of elimination of sources of pollution from the drainage areas, particularly the important drainage area of the Wachusett Reservoir, by diversion, purification or other adequate means. The Board made a sanitary survey of all the District's watersheds. In all unsewered areas, a detailed examination was made of every dwelling, farm, institution, industrial plant and summer camp, determining the number of residents or employees, amount of live stock, type and condition of sanitary facilities, method of waste disposal, and farm and barnyard drainage conditions, examining the water supplies, sewage disposal works, surface water filters, chlorinating plants, bathing beaches and swimming pools and refuse dumps.

Wachusett Watershed. The Wachusett watershed contains practically all of the populated areas of the towns of Boylston, West Boylston, Holden, Princeton and Sterling and part of Rutland. There have already been acquired for sanitary protection and for construction purposes 9,783 acres, or 15.3 sq. miles, of which 4,135 acres are flooded by Wachusett Reservoir. About one-fifth of the drainage area outside of the residential sections is devoted to farming and dairying. The remainder is woodland, varying from scrub, second-growth hard wood to occasional stands of merchantable pine. Twenty-three hundred acres of the total owned by the Metropolitan Water District have been reforested with pine. About 1,400 acres, under the control of the State Department of Conservation, are used for forestry purposes. The Wachusett Mountain State Reservation has a total area of about 1,570 acres, 380 of which are in the Wachusett watershed.

The Town of West Boylston is located very close to the shores of the reservoir, about 5 miles from its outlet at the dam. This town has not yet provided itself with a sewage disposal system. Such a condition will not exist on the new Quabbin Reservoir, where a real estate taking line has been established generally about 1 mile from the shore line and where a total of about 80,000 acres has been or will be acquired for sanitary protection and for construction purpose. One section of West Boylston, which will require immediate treatment, is Gates Brook, whose headwaters are in a recent and extensive real estate development in Worcester and West Boylston. This development consists principally of one and two family dwellings, many of which are surrounded by small gardens which extend to the property line of an all-too-narrow strip along the brook acquired by the District in 1916. This brook flows into the Wachusett Reservoir

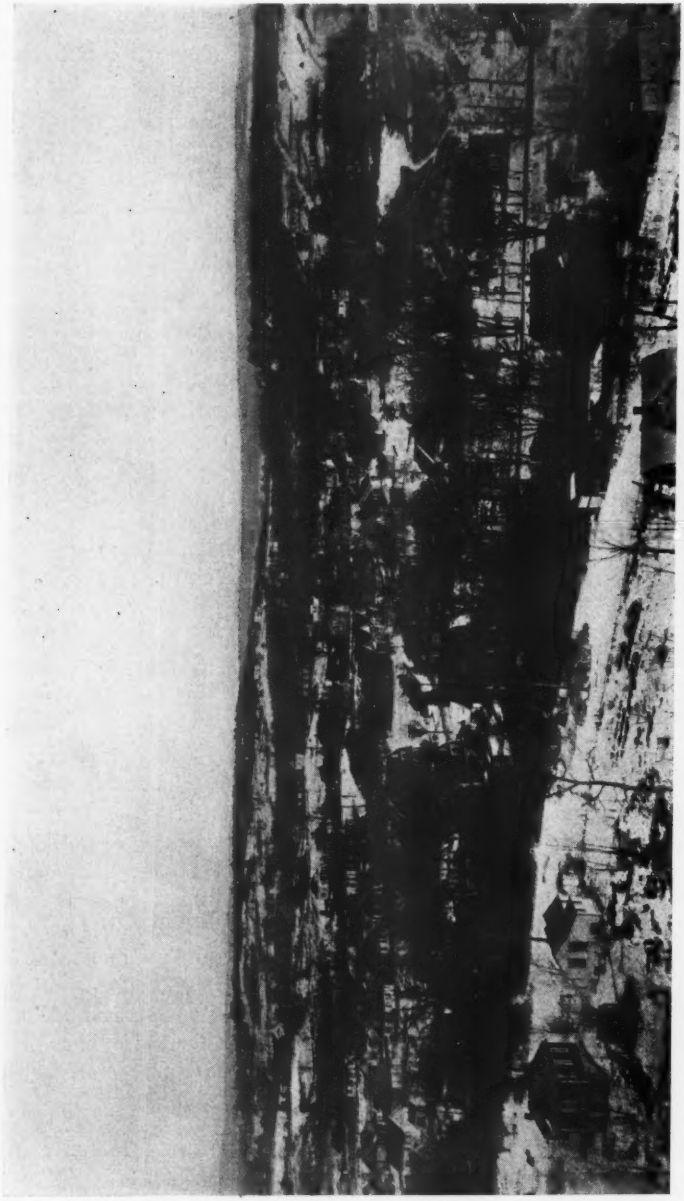


FIG. 4. GENERAL VIEW OF GATES BROOK AREA

through West Boylston. The population on its drainage area is about 446 persons per sq. mile, and samples of the water show a *B. Coli* index of 1500. See Figure 4.

The increase of population on the watershed has been gradual and hence has escaped the attention it deserves. The sanitary survey, however, shows many populated areas with unsatisfactory conditions of drainage. They should be eliminated without further delay. Works should be constructed to collect and dispose of the sewage and to treat other drainage.

In one instance in the Town of Holden, industrial wastes from a woolen cloth manufacturing and dyeing concern, amounting to about 300,000 g.p.d. when operating full time, are discharged directly into a tributary stream, a considerable distance from the reservoir to be sure, but nevertheless requiring immediate treatment.

The Wachusett Reservoir will continue for an indefinite period to be one of the principal sources of supply to the District, and the new supply from the Ware and Swift will pass through this reservoir and through the Wachusett Aqueduct. Accordingly, the elimination of sources of pollution in this area is one of the most important immediate steps to be taken. In spite of the 2,950 cattle and 1,365 pigs, to say nothing of 587,785 hens, on this watershed, Wachusett Reservoir has consistently delivered water of excellent quality. An analysis of water samples taken at different points in the watersheds and reservoirs and *en route* to the District indicates the startling amount of pollution picked up, as shown by the *B. coli* index (Plate II.) In other words, the long period of detention in Wachusett Reservoir has so far satisfactorily offset the effect of pollution from the Wachusett watershed, and the water delivered through the Wachusett Aqueduct is of the highest quality. It is badly abused and mistreated, however, before it gets to the consumer in the District.

Another index of the advantages of the more remote sources as well as an explanation of the added pollution from nearby areas is indicated in a comparison of the increase of population per square mile on the different drainage areas (Fig. 5.) After the area to be submerged has been vacated, there will be only 8 persons per square mile on the Swift watershed. There are 34 on the Ware watershed, 74 on the Wachusett, 550 on the North Sudbury, 230 on the South Sudbury and 1,225 on the Cochituate.

The exposed open channel through which the Wachusett Aqueduct discharges into Sudbury Reservoir, is responsible for a considerable part of the increase in pollution. This open channel discharges into the reservoir in the Town of Southborough, the populous center

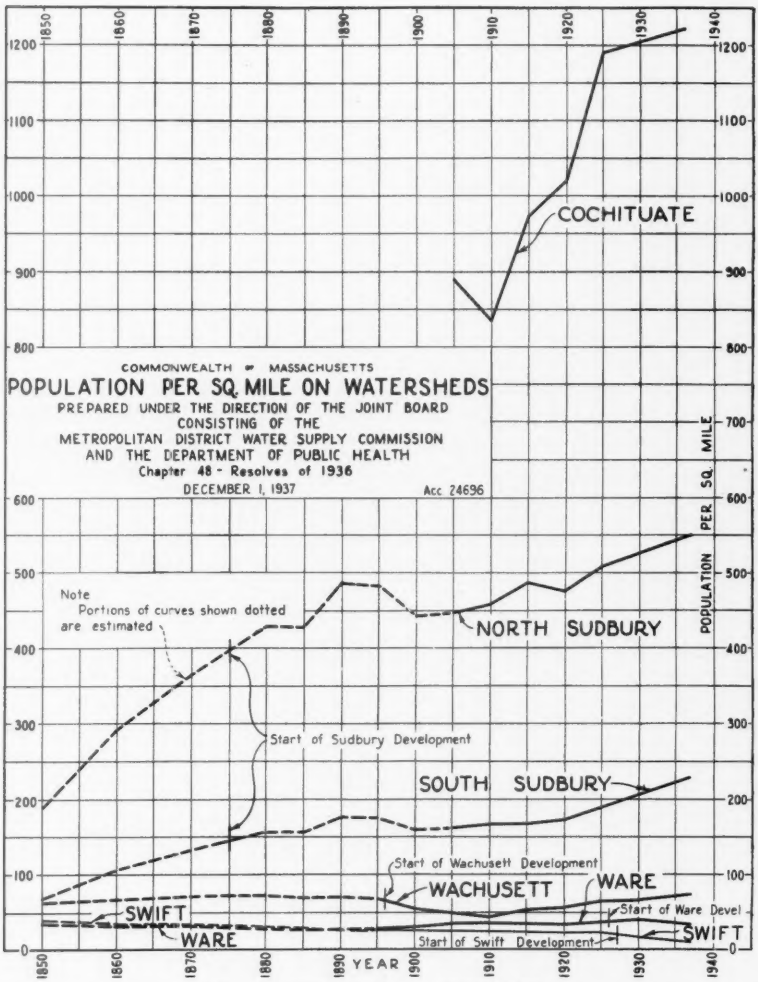


FIG. 5. POPULATION PER SQUARE MILE

of which is close to the shores of the Sudbury Reservoir (Fig. 6). The open channel forms the easterly part of Wachusett Aqueduct, and it has a tributary drainage area of 8.3 sq. miles above Deerfoot Road. A large sausage and milk pasteurizing plant is located very close to the reservoir, and although it has an independent sewage plant, it is not an ideal neighbor to the reservoir. Southborough Cemetery is located near the water's edge, but its contribution to the pollution is undoubtedly more imagined than real.

FIG. 6. AERIAL PHOTOGRAPH OF SOUTHBOROUGH

The open channel passes through an extensively developed farming region and is consequently subject to a considerable degree of both human and animal pollution. The drainage area of this channel has a population of 720 (87 persons per sq. mile) and has about 760 cattle pastured within its boundaries. Hence it is not surprising that the water in the lower end of the open channel has a high *B. coli* index.

North Sudbury Watershed. The North Sudbury watershed contains the principal parts of Marlborough, Southborough and a small part of Framingham (Fig. 7). Its population of 550 persons per sq.

FIG. 7. AERIAL PHOTOGRAPH OF MARLBOROUGH

mile makes it one of the most densely populated watersheds in Massachusetts used for water supply. The sanitary problem of preventing and controlling pollution from the Sudbury area is complicated by the diversified character of the premises within it, which consist of some 2,400 dwellings, numerous farms and truck gardens, 18 industrial establishments employing about 2,400 person at present, and several institutions. Although the principal residential, business and industrial sections of Marlborough are provided with a system of sanitary sewers, there are 160 premises within the sewered area still using private sanitary facilities, of which 30 are privies. There are several large dairy farms and estates with several hundred acres under cultivation or used for pasture.

Although steps have been taken to prevent any sewage from the Marlborough sewerage system being discharged into Sudbury Reservoir, the location of about 2 miles of main trunk sewer and 2 miles of secondary trunk sewer within the watershed below the area tributary to the Marlborough Brook and the Farm Street filters presents a very real hazard, as any overflow from these sewers due to breakage or other cause would allow raw, untreated sewage to flow directly into the reservoir. The Sudbury area also has a liberal scattering of privies. Southborough, located close to the reservoir, has no sewerage system.

The South Sudbury watershed contains the principal parts of Ashland, Hopkinton and Westborough and the villages of Southville and Cordaville in Southborough, also a part of Framingham. It has a population of 230 persons per sq. mile. There are some 1980 dwellings, many farms, and 21 industrial concerns employing about 1800 persons at present, on this watershed. The principal residential, business and industrial sections of Westborough are provided with a system of sanitary sewers which discharge into an intermittent sand filter plant outside the watershed. There are, however, about 180 premises within the sewered area not connected with the public sewer.

It is extremely unlikely, as soon as Quabbin Reservoir is completed, that the Southern Sudbury sources will again be required except that the pipe lines from Whitehall and Hopkinton reservoirs which discharge into Sudbury Reservoir, may be useful in emergencies.

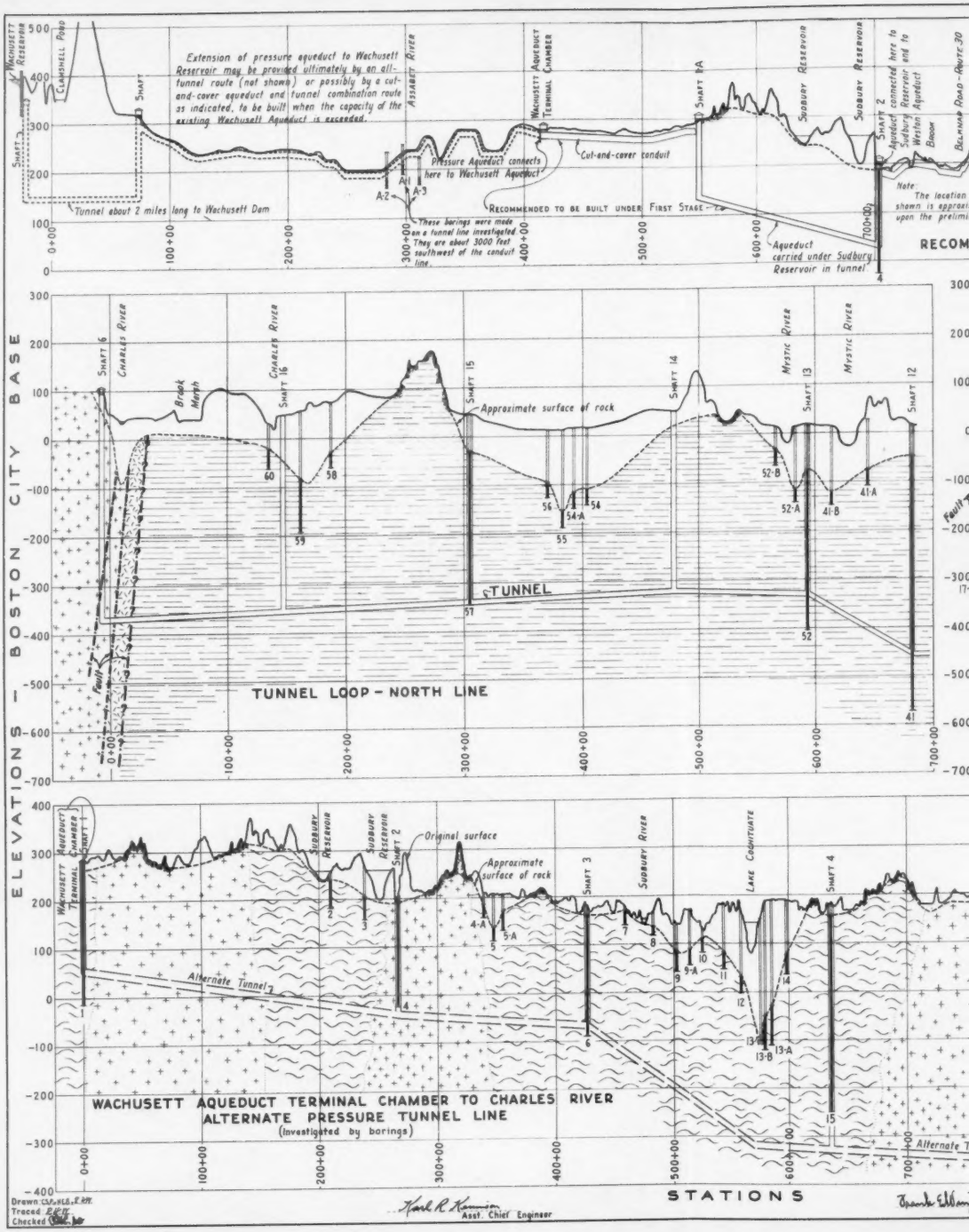
Lake Cochituate. As to Lake Cochituate, a full description and report on the sanitary condition of its watershed was made last year and published as House Document 293-1937 in accordance with Chapter 18 of the Resolves of 1936, which was the culmination of long continued pressure on the Legislature to discontinue this source and devote the lake to recreational uses. The Joint Board is now recommending that upon the completion of the first stage of the

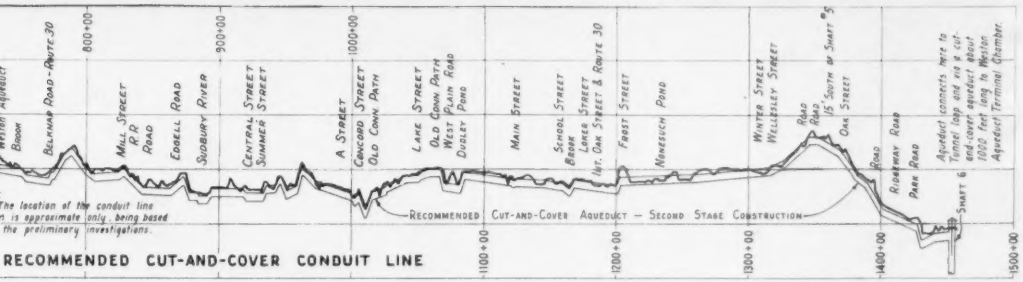
new construction which it recommends, as hereinafter explained, Lake Cochituate should be abandoned as a source of water supply.

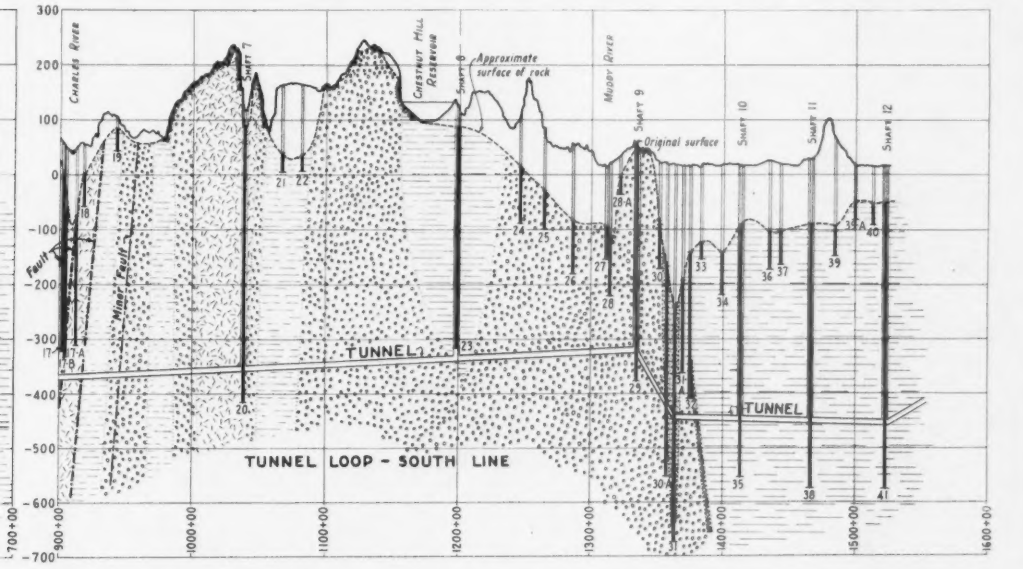
Incidentally the demand that water supply reservoirs be used for public recreation are vociferous and by no means confined to Lake Cochituate. I am told that in one of our neighboring states, the water supply authorities were shocked to discover that fishermen were living for days at a time in shelters they had erected actually on the ice surface of an important water supply reservoir. At a Massachusetts legislative public hearing several years ago, the late versatile Commissioner of Public Health, Dr. Bigelow, was asked if something could not be done to make it safe to permit fishing from the banks of Wachusett Reservoir. He replied that he knew of no way unless the permission could be restricted entirely to constipated fishermen. However, a practical remedy was reached last year when the Metropolitan District Commission made arrangements whereby the Department of Conservation each year seines the water supply reservoirs and places the fish in streams and ponds available to sportsmen.

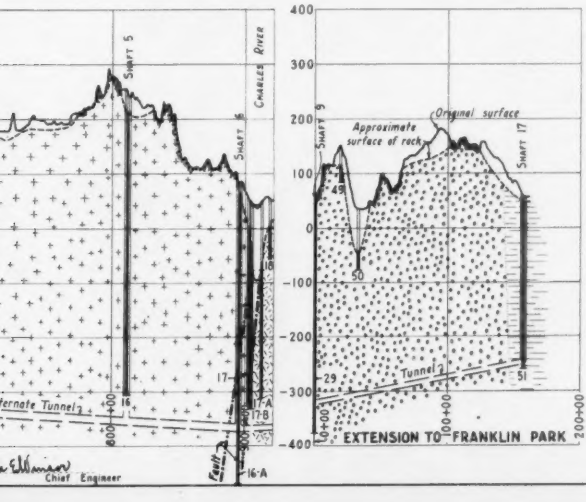
Existing Aqueducts

The District's supply is now delivered from the end of Wachusett Aqueduct as follows (See Plate I). After passing through the open channel in Southborough and through Sudbury Reservoir, El. 259 above Boston City Base, the supply comes to the District by two routes. There is not at present any reserve capacity in the distribution lines over and above the supply that might be consumed during a week of very heavy demand, which may reach a peak of 160 m.g.d. required to be delivered to the distributing reservoirs in the District which take care of the much greater daily and hourly peaks. Of this total 160 m.g.d., which the aqueducts are occasionally called upon to deliver in a maximum week, about 30 m.g.d. follows what might be referred to as the southerly low-level route: first about 2.5 miles through Basin No. 3, an open channel exposed to pollution along the shores; thence about 1.0 mile through two 48-in. cast-iron pipes from Dam No. 3 along the shores and the bed of Basin No. 1 to Dam No. 1; thence about 17.5 miles through the Sudbury Aqueduct to Chestnut Hill Distributing Reservoir, El. 134. The water is chlorinated at a point about 2.5 miles after it enters the aqueduct at Dam No. 1. This aqueduct is a grade-line aqueduct, 17.38 miles long, completed in 1878 by the City of Boston. It is not sufficiently tight to insure the exclusion of ground water where this is higher than the water level in the aqueduct, and there are many known leaks in Framingham, Natick and Wellesley where polluted water may enter


Extension of pressure aqueduct to Wachusett Reservoir may be provided ultimately by an all-tunnel route (not shown) or possibly by a cut-and-cover aqueduct and tunnel combination route as indicated, to be built when the capacity of the existing Wachusett Aqueduct is exceeded.
Wachusett Reservoir
Clamshell Pond
Shaft 1
Shaft
Tunnel about 2 miles long to Wachusett Dam
Assabet River
Wachusett Aqueduct Terminal Chamber
Pressure Aqueduct connects here to Wachusett Aqueduct
A-1
A-2
A-3
These borings were made on a tunnel line investigated. They are about 3000 feet southwest of the conduit line.
Cut-and-cover conduit
Recommended to be built under First Stage
Shaft 1A
Sudbury Reservoir
Sudbury Reservoir
Shaft 2
Aqueduct connected here to Sudbury Reservoir and to Weston Aqueduct
Boston
Belmont Road - Route 30
Aqueduct carried under Sudbury Reservoir in tunnel
Note: The location shown is approxi upon the prelimi
RECOM
ELEVATIONS — BOSTON CITY BASE
Shaft 16
Charles River
Brook Marsh
Charles River
Shaft 15
Approximate surface of rock
Shaft 14
Mystic River
Shaft 13
Mystic River
Shaft 12
TUNNEL
Fault
TUNNEL LOOP - NORTH LINE
Original surface
Approximate surface of rock
Alternate Tunnel
Shaft 3
Sudbury River
Lake Cochituate
Shaft 4
WACHUSETT AQUEDUCT TERMINAL CHAMBER TO CHARLES RIVER
ALTERNATE PRESSURE TUNNEL LINE
(Investigated by borings)
STATIONS
Drawn C.S.P., H.L.B., E.R.H.
Traced E.R.H.
Checked
Hank R. Kennison
Asst. Chief Engineer







~ Legend ~

Granite and Tonalite
Schist
Shale and Argillite
Volcanics
In earth overburden
In rock
Conglomerate
Cretaceous or Tertiary Shales and Sandstone
Tillite
35 Boring number

Notes:
The location and depth of the tunnel and shafts are approximate only being based upon the preliminary investigations made for this report.
The profiles show generally the geological distribution of the rock formations investigated, although the details are approximate only and proportions are distorted because of the exaggerated vertical scale. No attempt has been made to show geologic structure on this sheet; see geologic sections.

Chief Engineer

COMMONWEALTH OF MASSACHUSETTS

JOINT BOARD CONSISTING OF
METR. DISTR. WATER SUPPLY COMMISSION
AND
DEPARTMENT OF PUBLIC HEALTH

CHAPTER 48 - RESOLVES OF 1936

NEW PRESSURE AQUEDUCT FOR METROPOLITAN SUPPLY

PROFILES

DECEMBER 1, 1937.

Acc. 24688

FIG. 8. LEAK IN SUDBURY AQUEDUCT

FIG. 9. LEAK IN SUDBURY AQUEDUCT

the aqueduct. The liability to pollution from leakage into these old grade-line aqueducts has been shown by an inspection of Cochituate Aqueduct (See Figs. 8 and 9). This particular aqueduct is not now in regular use. It may have to be put into use at any moment if either the Weston or Sudbury Aqueducts have to be closed for repairs or otherwise. Chestnut Hill Reservoir, at the outlet of Sudbury Aqueduct, offers further possibilities of pollution along its exposed shores. The water at Chestnut Hill Reservoir is then again chlorinated and raised 117 ft. by pumping, at the Chestnut Hill pumping station in Boston, to the Fisher Hill distributing reservoir of the southern high-service, El. 251, where it is again chlorinated, and supplies Quincy, Milton, Hyde Park, Roxbury, Dorchester and other parts of Boston, through a network of large surface mains, up to 54 in. in diameter.

A small portion of this water, about 2 m.g.d., is pumped a second time at the Hyde Park pumping station to the Bellevue standpipe of the southern extra-high-service, El. 375.

In addition to this pumpage to Fisher Hill Reservoir, a small portion of this low-level supply is chlorinated and pumped from Chestnut Hill to the Waban Hill Distributing Reservoir of the southern high service in Newton, El. 264.5, where it is again chlorinated and from which most of the supply now goes to Watertown. All the pump intakes at the Chestnut Hill Station are equipped for emergency chlorination.

The balance of the required supply in the maximum week, namely 130 m.g.d., leaves the Sudbury Dam via the Weston Aqueduct, about 11.3 miles to the Weston Distributing Reservoir, El. 200. This aqueduct also is a grade-line aqueduct, completed in 1903 by the District. It was designed to carry about 300 m.g.d., but at each of the inverted siphons, so-called, about 3610 ft. across the Sudbury River valley and about 1125 ft. across Happy Hollow, there is only a single line of pipe, with provision for future additions. One of the recommendations of the Board is that an additional pipe line be immediately constructed in each of the two inverted siphons on the Weston Aqueduct, which will permit the entire aqueduct to carry its full capacity. Until these siphons are completed, the capacity of the aqueduct is only about 150 m.g.d. This aqueduct is the most important present link in the distribution system. It is well constructed, and there is little risk of pollution from surface infiltration of ground water. The exposed shores of Weston Reservoir, comprising an open channel 1.0 mile long to the screen chamber, are, however, possible sources of pollution, since the ditches for diversion of local drainage are not, under some conditions, entirely effective. From Weston Reservoir screen cham-

ber, where the supply is chlorinated, the aqueduct extends another 1.1 miles to the Weston Aqueduct Terminal Chamber.

From the Weston Aqueduct Terminal Chamber three routes are followed, all in large mains. About 60 m.g.d. are carried by two cast-iron pipes, one 60 in. and one 48 in. in diameter, along Commonwealth Avenue to Cleveland Circle; thence in three 48-in. cast-iron pipes to the low service in Boston. The 60-in. and 48-in. pipes also discharge at times into Chestnut Hill Reservoir, to supplement the flow from Sudbury Aqueduct and supply the southern high-service pumps, which handle about 35 % of the total consumption of the District. About 40 m.g.d. are carried from the Weston Aqueduct Terminal Chamber by a 60-in. steel pipe via Washington Street in Newton and the Metropolitan Parkway along the Charles River to Allston and supply more of the low service. About 30 m.g.d. are carried from the Weston Aqueduct Terminal Chamber by a steel pipe, ranging from 56 to 60 in. in diameter, through Waltham, Belmont and Arlington to Medford and thence through a cast-iron pipe, 48 in. to 60 in. in size, into the distribution system in the vicinity of Spot Pond, El. 163.

A small portion, about 2 m.g.d., of the flow through this pipe will, after certain changes now planned are made effective, be pumped at the Belmont pumping station into the new Arlington Reservoir of the intermediate high-service, El. 320; and another small portion, less than 2 m.g.d., will be pumped at the Arlington pumping station into the Arlington standpipe, El. 442.5.

Spot Pond, into which this pipe from Weston Aqueduct discharges, is surrounded by Metropolitan Park areas with a menagerie, bridle paths and many public roads, and hence is particularly difficult to protect from pollution. At the Spot Pond Pumping Station, the supply is again chlorinated and about 10% of the total supply is raised 108 ft. and 137 ft. by pumping to the Fells and Bear Hill distributing reservoirs of the northern high service, El. 271 and 300, respectively.

These reservoirs, particularly the former, should be protected by adequate fencing. The line from Weston Aqueduct Terminal Chamber to Spot Pond is also connected with the low-service network north of Boston, which under heavy draft draws its supply directly from Spot Pond, the draft from the east outlet at the pumping station being chlorinated. Hence Spot Pond, with its 1800 m.g. storage capacity, serves the low-service distribution system and is the most important present distribution reservoir in the system.

In order to reduce the effect of extreme fluctuations in the rate of demand from hour to hour throughout the day and to prevent too high pressures, particularly in the low-service pipes in Boston proper,

during hours of low demand, (1) the rate of discharge through Weston Aqueduct is varied by the regulating gates at Sudbury Dam to as great an extent as is feasible, and, (2) the discharge into the Chestnut Hill Reservoir from the Weston Aqueduct pipe lines is also varied during the day by the regulating gates at Chestnut Hill.

Under the present plan of operation, there is practically no surplus capacity in the distributing mains from Weston Terminal Chamber during a week of heavy draft, 20% in excess of the yearly average. Under average conditions, such surplus as exists is controlled by throttling the 60-in. main to Spot Pond. Unless and until another 60-in. main is built, the only way in which additional capacity can be obtained, is by the poorly protected low-level route previously described, that is, by taking more water through Sudbury Aqueduct. Any sudden increase in the low-service demand will make it necessary to pump this Sudbury Aqueduct supply into the low-service mains. Three low-service pumps at Chestnut Hill are kept in readiness for such a call, and steam is kept on the boilers. West of the Weston Aqueduct Terminal Chamber, if there should be any interruption of the flow in the Weston Aqueduct, the District could be supplied for three or four days from the reserve storage in Spot Pond and Chestnut Hill reservoirs. Any longer interruption is improbable but if it should occur, the flow in the Sudbury Aqueduct could be increased to its full capacity, probably little, if any, over 100 m.g.d., and in addition the old Cochituate Aqueduct and its emergency source could be used, at a maximum rate of 23 m.g.d. This would make a total supply to the Chestnut Hill pumps of not over 125 m.g.d. entering the District in such an emergency.

Briefly stated, the future water supply of the District will come from sources remarkably free from pollution. The water as drawn from long-term storage in Wachusett Reservoir will continue to be of excellent quality. However, between the Wachusett Aqueduct Terminal Chamber, where the supply is discharged into an open channel, and the pump intakes at Chestnut Hill Pumping Station and Spot Pond Pumping Station, a distance of roughly 25 miles, the present means of distribution are particularly vulnerable from the standpoint of quality. This situation can be satisfactorily remedied by eliminating the Sudbury Reservoir and Aqueduct system from regular use, and its retention as an emergency reserve only, in a rôle similar to that now played by the Lake Cochituate System; and by eliminating from regular use the low-level distributing reservoirs at Chestnut Hill and Spot Pond, which are practically impossible adequately to protect from pollution. I cannot emphasize too strongly the need of immediate steps to remedy this situation.

The situation is equally, if not quite, as bad from the standpoint of carrying capacity.

The cities and towns of the District are scattered over a wide, fairly level area resembling the bottom of a bowl, and it is only in the surrounding ring of hills that adequate high-service distributing reservoirs can be found; for example, in the Middlesex Fells, in Weston and in the Blue Hills. In addition, high ground near Chestnut Hill furnishes several meager sites. As a result, the cities and towns in the District are supplied through a network of surface mains which extend to great lengths between the distributing reservoirs and the points of consumption.

Furthermore, these mains have to cross the natural barriers of the Charles and Mystic rivers. Their great lengths and crowded condition in the highways make for extreme vulnerability to breaks, leaks, long shutdowns for repair, etc., and even those areas which can normally be supplied by gravity require, on account of the fire risk, the maintenance of the pumps in continual readiness for operation to furnish adequate pressures. There are already three 60-in. mains and one 48-in. main fed from the Weston Aqueduct Terminal Chamber. Without a radical change in the plan of development, the District faces the necessity of constructing a new 60-in. pipe line in the next year or so and one about every six or seven years thereafter. As soon as the urgently needed additions are made to the Weston Aqueduct inverted siphons, its capacity will be increased to its designed 300 m.g.d. When it becomes necessary to deliver this quantity to the District, if the delivery had to be made through 60-in. mains as at present, about six or seven such mains would be required.

Recommended Improvements

The Joint Board recommends a plan for the future distribution of water which will eliminate the multiplicity of large mains in the streets, reduce to a minimum the hazards of breakage, maintain the desired pressures under conditions of extraordinary draft due to conflagration or other causes, and insure the delivery of water from the best upland sources with no contamination or pollution *en route*.

New Aqueducts. The problem can best be solved by conducting the water in pressure aqueducts for the entire distance from the Wachusett Aqueduct Terminal Chamber to the points of consumption, the pressure aqueduct being eventually extended to Wachusett Dam. This plan has the added advantage that practically all pumping now required will be eliminated. The 5 or 6% of water still requiring pumping to the extra-high-services will have the benefit of a reduction in the head pumped against.

The most practicable plan for constructing pressure conduits in the District proper is by a tunnel driven deeply through the underlying rock. Geological studies with the aid of core borings along a projected line have demonstrated the feasibility of this plan (See Plates III and IV). In the outlying areas, also, west of the Charles River, the geological studies and borings have demonstrated the feasibility of tunnel construction for the entire distance. However, progress in the art of construction of large pressure conduits of steel and concrete has been rapid in recent years, and the economics of the situation justify the use, in portions of the line, of large surface aqueducts similar to those recently constructed in portions of the Colorado River Aqueduct for the supply of the Metropolitan Water District of Southern California.

The terminal distributing reservoir in the future system would be an enlargement of the present Middlesex Fells Distributing Reservoir. A future elevation of 268 ft. at this point will, under the improved conditions of tunnel distribution, satisfactorily serve all the cities and towns now supplied by the northern and southern high services. This elevation is 10 ft. lower than the flow line of the Wachusett Aqueduct Terminal Chamber. Hence the supply will be delivered by gravity, and adequate outlets into the distribution system at each shaft would be provided.

The diameter of such a pressure aqueduct from Wachusett Aqueduct Terminal Chamber to the Metropolitan District which will best fit both present and future requirements does not exceed 15 ft. An aqueduct of this diameter will continue to be adequate until the maximum rate of demand within the District is 350 m.g.d. This equals approximately the capacity of the present Wachusett Aqueduct through which the supply is carried to the Terminal Chamber. The aqueduct is designed so that in the more remote future, it can be extended westerly to Wachusett Dam, and so that sufficient additional head can then be put on the entire line to the new high-level distributing reservoirs to increase its capacity from 300 to 600 m.g.d., a peak-load figure appropriately consistent with the aqueduct capacity provided west of the Wachusett Dam.

The loop tunnel within the District proper, by which the water can be supplied to any point from two directions, is an ideal solution to the problem of emergency tunnel unwatering without interference with gravity delivery. Hence the tunnel loop will permit the dismantling of all pumping stations except those small auxiliary stations which will be used to pump about 5 or 6% of the supply to the extra-high-service areas in parts of West Roxbury, Hyde Park, Milton, Brookline, Newton, Stoneham, Arlington and Belmont. Fur-

thermore, the reduction of tunnel diameter, made possible by the loop system, is particularly advantageous, because that section of the tunnel required to be built as the first stage of the development may be materially reduced in diameter and still satisfactorily meet present requirements. The tunnel loop and the pressure aqueduct, or tunnel if then desired, paralleling the Weston Aqueduct can later be completed to obtain the ultimate carrying capacity and pressure, about fifteen years later when the cost can be more easily financed. With one unwatering shaft near the Weston Aqueduct Terminal Chamber discharging into the Charles River and another on the waterfront in Charlestown discharging into Boston Harbor, it will be possible to unwater any one of four sections of the loop without any interruption of service or without material drop in pressure. Otherwise it would be necessary to maintain the Chestnut Hill and Spot Pond high-service pumps for emergency operation.

A branch tunnel should also be extended from the loop to the south to supply heavy demands in the southern communities and to reach a point within striking distance, *via* large mains, of a distributing-reservoir site in the Blue Hills. There is in the Blue Hills reservation, on land already controlled by the Metropolitan District Commission an excellent site for such a reservoir. It should be constructed as soon as the needs of the District in the more remote future make it economically desirable to extend large mains to this point. The site can be developed by the construction of a dam of moderate height so that it will provide emergency storage for as much as 350 m.g.

The Joint Board specifically recommends that in order to by-pass Sudbury Reservoir and the open channel into it, a section of the pressure aqueduct about 5½ miles long be immediately constructed from the Wachusett Aqueduct Terminal Chamber to the Sudbury Dam, having connections at the latter point with the Weston Aqueduct.

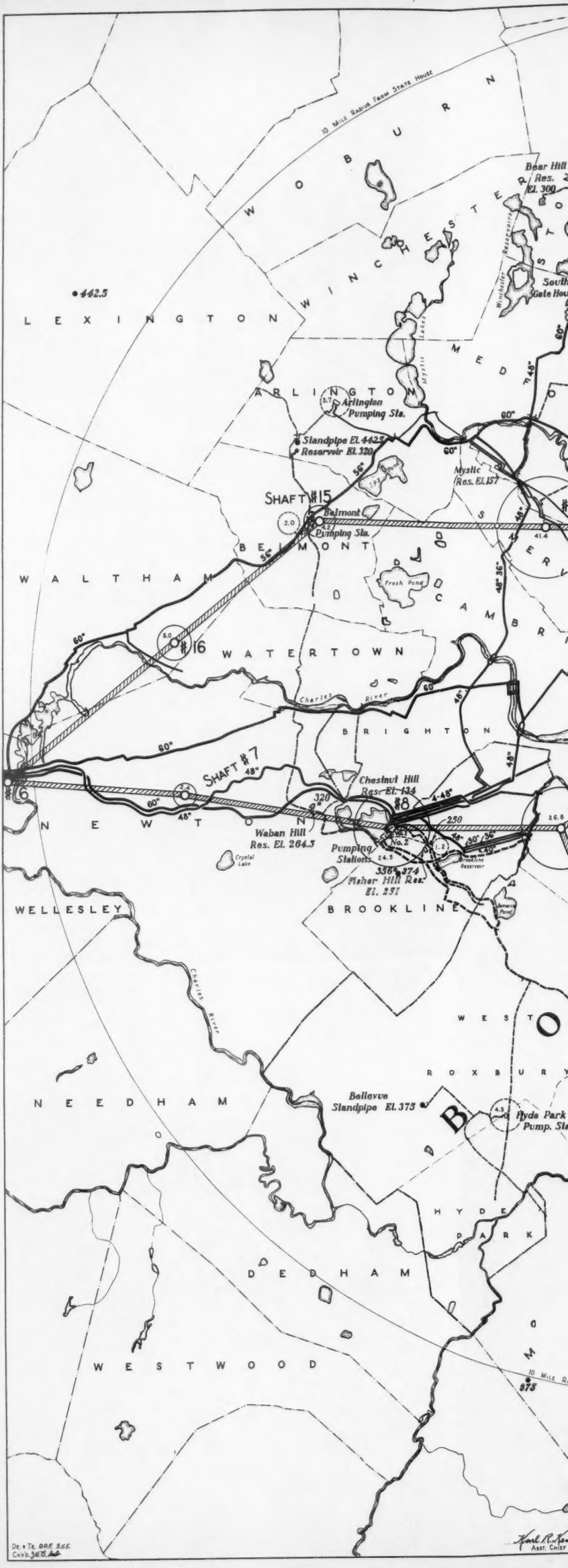
This section will consist of a steel and concrete cut-and-cover pressure conduit, about 2½ miles long, in the city of Marlborough and the town of Southborough, from the Wachusett Aqueduct Terminal Chamber to a tunnel shaft. Thence under Sudbury Reservoir about 3 miles to the Weston Aqueduct at Sudbury Dam, the aqueduct will be built as a pressure tunnel about 14 ft. in diameter. It may be desirable to build the cut-and-cover portion of this section of the aqueduct of a diameter smaller than the 15 ft. which would otherwise be ultimately required and to increase the capacity by a parallel line when needed.

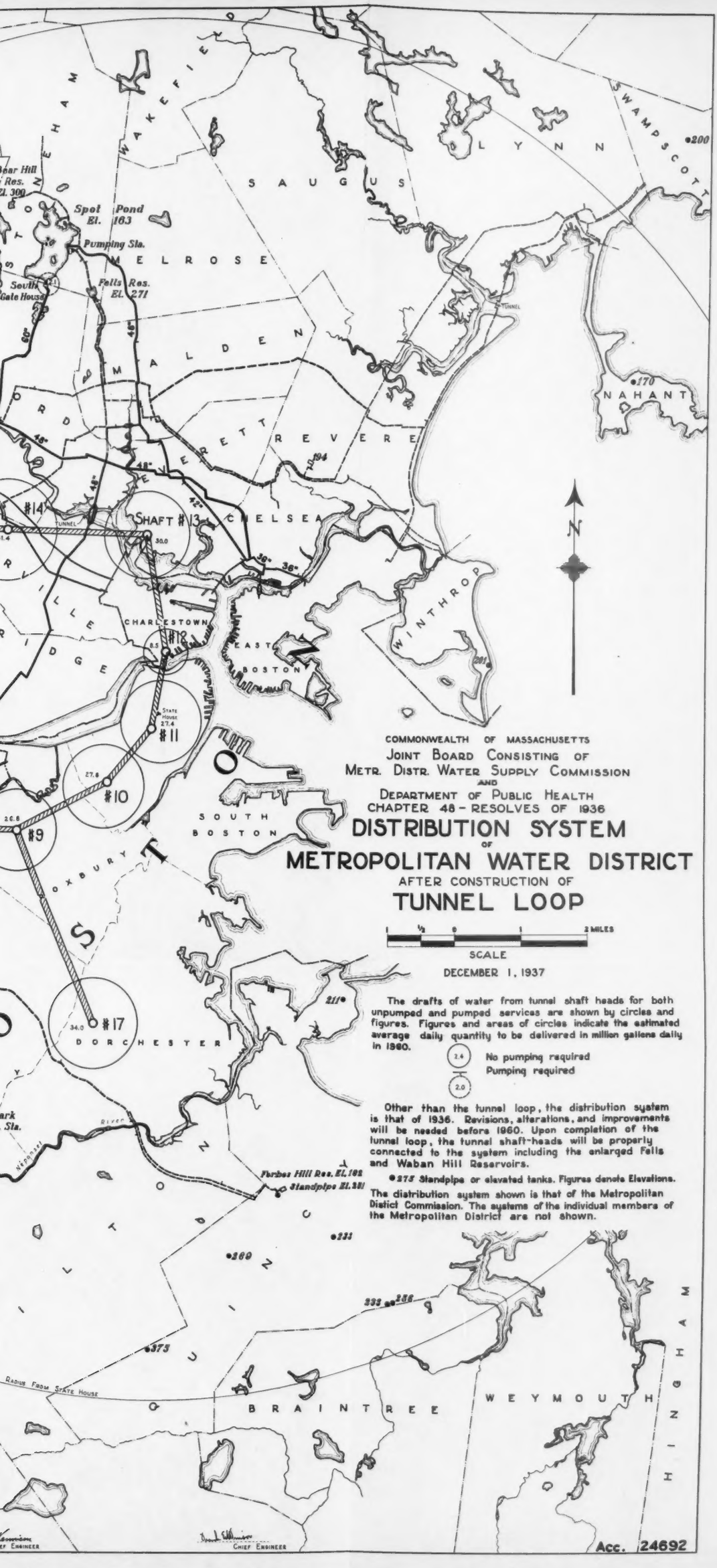
The Board also recommends that in order to permit the entire supply to be delivered through Weston Aqueduct and thus to permit discontinuing the use of Sudbury Aqueduct and Chestnut Hill Res-

ervoir, a section of the tunnel loop about 5¾ miles long between a point near the Weston Aqueduct Terminal Chamber and a point near the Chestnut Hill pumping station be immediately constructed (Plate V). This first section of the loop will alter and improve the conditions of distribution in the District to such an extent that, with certain other minor changes recommended, including 3500 ft. of pipe line by-passing Spot Pond, the regular use of Spot Pond also can be discontinued. This important distributing reservoir will, however, have to be held in reserve and in readiness for use until such time as the entire program of high-pressure delivery through the completed tunnel loop can be put into operation. The estimated cost of the proposed first stage of construction is $12,000,000.

In this connection, it is worth noting that material savings have been effected in the construction of the extensions to the Ware-Swift supply during the last 11 years in spite of a considerable amount of additional work which the Legislature has given the Metropolitan District Water Supply Commission to do and pay for out of the original appropriation. The amount saved from these appropriations is fortunately sufficient, if made available, to do the work which the Joint Board now recommends as a first step in the urgently needed program for sanitary protection of the water supply and improvement of the distribution system. It is estimated that completion of the entire program outlined can be postponed for not more than about 15 years.

Cost of Water. The program laid out for the development is such that it will not further materially increase the cost of water delivered to the cities and towns in the District. As a matter of fact, it is estimated that the cost per million gallons will soon show substantial reductions from the present figure of about $100. Figure 10 indicates the extent to which the costs have increased since 1925. Only about two-thirds of this increase is due to the construction of the extension to the Ware-Swift supply, financed by serial bonds limited to 30 years, the other third being due to the adoption by the State in 1925 of a pay-as-you-go policy in constructing necessary extensions to the distribution system. Figure 10 also indicates the extent to which costs will be reduced in the future regardless of the improvements by pressure aqueduct construction recommended by this Board. This inevitable reduction is due not only to the pending completion of the Ware-Swift extension, but also to the fact that the original construction of the Wachusett-Sudbury system was financed by 40-year sinking fund bonds almost all of which will soon be entirely paid off. One effect of the anticipated reduction in cost per million gallons will be to make membership in the District attractive to many


10 Mile Radius From State House
WOBURN
WINCHESTER
LEXINGTON
ARLINGTON
BELMONT
WALTHAM
WATERTOWN
CAMBRIDGE
BRIGHTON
NEWTON
WELLESLEY
BROOKLINE
NEEDHAM
DEDHAM
WESTWOOD
HYDE PARK
ROXBURY
Mystic Lakes
Winchester Reservoirs
Charles River
Fresh Pond
Crystal Lake
Arlington Pumping Sta.
Standpipe El. 442.5
Reservoir El. 320
Mystic Res. El. 157
SHAFT #15
Belmont Pumping Sta.
SHAFT #7
#16
#6
#8
Chestnut Hill Res. El. 134
Waban Hill Res. El. 264.3
Pumping Stations
No. 2
336-374
Fisher Hill Res. El. 251
Bellevue Standpipe El. 375
Hyde Park Pump. Sta.
Bear Hill Res.
El. 300
South Gate Hou
442.5
375
10 MILE R



COMMONWEALTH OF MASSACHUSETTS

JOINT BOARD CONSISTING OF
METR. DISTR. WATER SUPPLY COMMISSION
AND
DEPARTMENT OF PUBLIC HEALTH
CHAPTER 48 - RESOLVES OF 1936

DISTRIBUTION SYSTEM OF METROPOLITAN WATER DISTRICT

AFTER CONSTRUCTION OF

TUNNEL LOOP

1 ½ 0 1 2 MILES

SCALE

DECEMBER 1, 1937

The drafts of water from tunnel shaft heads for both unpumped and pumped services are shown by circles and figures. Figures and areas of circles indicate the estimated average daily quantity to be delivered in million gallons daily in 1960.

(2.4) No pumping required

(2.0) Pumping required

Other than the tunnel loop, the distribution system is that of 1936. Revisions, alterations, and improvements will be needed before 1960. Upon completion of the tunnel loop, the tunnel shaft-heads will be properly connected to the system including the enlarged Falls and Waban Hill Reservoirs.

●275 Standpipe or elevated tanks. Figures denote Elevations.

The distribution system shown is that of the Metropolitan Distict Commission. The systems of the individual members of the Metropolitan District are not shown.

CHIEF ENGINEER

CHIEF ENGINEER

Acc. 24692

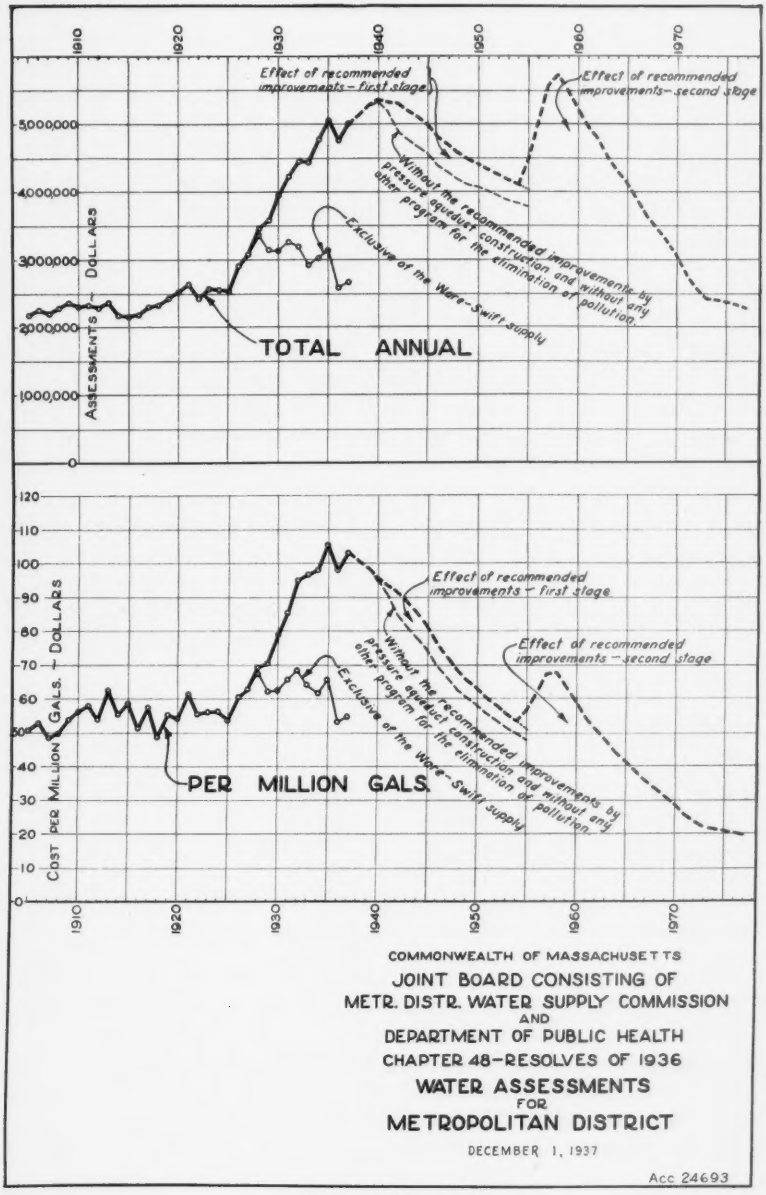


FIG. 10. ASSESSMENTS

of the 15 municipalities not now members of the Water District but eligible to membership, since they are within 10 miles of the State House. I recognize the fact that the size of the entrance fee is an important factor in determining whether or not a community may desire to join the District. The entrance fee should be based primarily on the fact that the new member is buying a share in the works and recognition should also be made that the new member is necessarily assuming the obligation to help to pay the outstanding indebtedness.

To determine such a proper basis for entrance fees, it is necessary to determine the present depreciated value of the works of the Metropolitan Water District. Some parts of the system have served a useful purpose in the past but are of little value as parts of the works planned for the future. The total capital cost of the Metropolitan Water Works to date, including unexpended balances from current appropriations and bond issues, is $94,547,403. It is estimated that the depreciated value of these works is about $80,010,450. About 56% of the total amount will have been paid for on April 1, 1938, leaving a net bonded debt equal to $41,285,000. Subtracting this Metropolitan water debt from the total depreciated value of the works leaves $38,725,450 as the estimated net value of the works, that is the amount paid for out of assessments to and including that of 1937 by the cities and towns in the District (See Table 3).

TABLE 3—APPROXIMATE NET WATER DEBT, AS OF APRIL 1, 1938

METROPOLITAN DISTRICT COMMISSION	ORIGINAL DEBT	PER CENT PAID	NET DEBT
Original Construction with Sinking Fund Bonds	$41,398,000	91	$3,700,000
Subsequent Construction with Serial Bonds	4,287,000	46.6	2,289,000
Construction with Current Funds and Short Term Notes	5,612,403	100	0
METROPOLITAN DISTRICT WATER SUPPLY COMMISSION			
Construction with Serial Bonds	42,250,000	16.5*	35,296,000
Construction with Current Funds	1,000,000	100	0
Totals	$94,547,403		$41,285,000

*Actually about 18.2% of these bond issues will be paid for as of this date because $720,000 will be paid on January 1, 1938, in advance of its assessment, in 1938, against the cities and towns in the District.

It should now be recognized that it is the enlargement of the works and the improvements in the distribution system which make it possible to increase the membership and that new members come into the District to enjoy the benefits as well as to share the obligations of the enlarged system. Accordingly, after these improvements have been completed, entrance fees should be made as small as pos-

sible in keeping with an equitable sharing of the depreciated value of the works, encouraging new members to join. Not only is each new member benefited, but also, since the member shares in the distribution of the total assessments, the cost per million gallons of water consumed by every member is reduced somewhat. In a like manner it would prove beneficial to all if eligibility to the present District were extended to communities located outside the ten-mile limit, and the Joint Board has so recommended.

METHODS USED TO FINANCE MAIN PIPE EXTENSIONS IN THE HARTFORD, CONNECTICUT, DISTRIBUTION SYSTEM

BY WARREN A. GENTNER*

[*Read February 17, 1938.*]

Three general methods have been employed to finance ordinary extensions to the distribution piping system at Hartford, Conn. The purpose of this paper is to review briefly each of these methods, and to discuss somewhat more fully the application and results obtained from the so-called "assessment" method.

Under the first plan (effective from the beginning of operations in 1853), all requested extensions to the piping system were financed from water revenue, augmented when necessary by a general tax levy on the city grand list. With the growth of the city and its extension into hitherto sparsely settled areas, requests for extensions became more numerous, and the problem of bringing water service to these scattered, semi-rural, small revenue-producing areas placed an increasingly heavy burden on the water department's financial structure. To relieve this drain on financial resources and at the same time to curb the number of requests for extensions for which the needs were least urgent, the Common Council on March 11, 1861, approved a resolution "authorizing an extension of street water mains whenever an annual income of ten (10) per cent. on the expense is guaranteed by those who ask for such extension."

Under this plan, persons desiring an extension of water service signed an agreement guaranteeing an annual income in water rents, or otherwise, equal to 10% of the cost of installing a pipe line from the nearest existing water main to the property for which service was desired. Water rents received from connections made to the extension were figured as part of the guaranteed income, and the difference between one-tenth the cost of the pipe line and the sum of all water rents was the amount which the guarantors paid annually out of their own pockets.

The guarantee method succeeded insofar as it shifted a portion of the financial responsibility for main pipe extensions to the consumer directly benefited, and at least temporarily reduced a large proportion of the more unwarranted requests for extensions. In the long run, however, the plan was unfair to both guarantor and water

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utility. It was unfair to the guarantor primarily because, being of the nature of a joint and several obligation, each signer was held responsible by the water department for payment of the guaranteed income, there being no provision for proportioning this responsibility over all consumers receiving their supply of water from the extension. It was unfair to the water department because of failure to provide for continuity of payment of the annual guarantee in the event of change in ownership of the guarantor's property, or his removal from the city, or death.

Despite its defects, this method continued in use for 61 years and was not abandoned until 1922, when as a result of increasing dissatisfaction with its operation, the Board of Water Commissioners proposed a method of assessing the cost of main pipe installations against abutting properties, a plan which was approved as a city ordinance by the Court of Common Council and became effective January 24, 1922.

Experience and results obtained from the early application of this plan of financing main pipe extensions in the Hartford system were discussed in papers presented before the American Water Works Association in May, 1923, and before this Association in September 1925 by Caleb Mills Saville, Manager and Chief Engineer of the Water Bureau of the Metropolitan District of Hartford County. The legality of this form of assessment is well established, as evidenced by several cases cited in Mr. Saville's papers.

In its original form, the Hartford assessment plan provided for the following:

(a) a basis for assessing the cost of mains, by using the average cost of 6-in. pipe lines for the previous year, providing this average cost did not exceed the actual cost of the main to be assessed;

(b) the amount to be assessed against each foot of property frontage equalled the total cost of laying the main, divided by the total frontage;

(c) the assessment was payable in full on completion of the work, or could be extended over ten equal annual payments, plus interest at 6% per annum on the unpaid balance;

(d) all assessments became liens against the property assessed, from the date on which laying of the pipe was authorized; and,

(e) exemption up to 75 ft. on corner lots, with water mains already in service along the other frontage.

Since the adoption of the plan, radical changes have been made in practically all these basic features. In the very first year of the assessment plan's operation (1922), Hartford, in common with other cities, experienced a brisk revival in building construction and real estate development activities, resulting in a very considerable increase

in demand for main pipe extensions and necessitating the laying of long lengths of pipe by machine in unpaved streets at much lower cost than the usual handwork in city streets. As a result of machine work, the average cost of laying pipe in 1922 was too low to use as a basis of assessment for work during the following year, when handwork would ordinarily predominate, without placing an unfair burden on the department. To rectify this condition, the plan was amended in April 1923, by changing the basis of assessment from the average cost of laying 6-in. pipe during the previous year, to the average cost for the previous five years.

In March 1925, the method of computing the cost per foot of frontage was changed by applying the total cost of the job to the total frontage less exemptions, instead of the total gross frontage. Under the original procedure, the water department assumed the expense due to exemptions and street intersections. In the revised form, this cost was spread over the remaining assessable frontage. In 1929, the 75-ft. exemption on corner lots was increased to 125 feet. In 1930, the basic size of pipe was changed from 6 to 8 in., due to the discontinuance in the use of 6-in. pipe, except in very short streets, or as short filler sections at the center of 8-in. lines of moderate length.

The period of deferred assessment payment was shortened from ten to three years in 1930 to reduce the amount of water department funds, which the longer term payment plan tied up in main pipe installations, and thus to make the funds more frequently available for this work.

At the present time, payments may be made in full within thirty days of the completion of the work, or they may be divided into four equal payments, the first being due and payable within a period of thirty days of written notice that the work is completed; the second, third, and fourth instalments being payable within two, three and four years respectively from the date of the notice, with 6% interest on the unpaid balance.

The routine procedure followed in the consideration of extensions under this plan is briefly as follows:

Proponents of an extension are first required to file with the Bureau a signed petition, stating the street limits to be covered by the extension, their names and addresses, and property owned by them, which must be located along the line of the proposed extension. On receipt of this petition, the town authorities in which the street is located are requested to furnish information as to whether or not the street is fully accepted and rough-graded, for unless these requirements are complied with, the water main will not be laid. A map, showing street lines, property lines, names of property owners with

frontage, and owners of abutting properties, is next prepared from the official city or town maps and records, and from this map a list of property owners and a tentative schedule of assessments are prepared.

The property list is sent to the Registrar, who sends written notice of a hearing (to be held for the purpose of determining the public necessity for the proposed extension) to each property owner along the proposed extension, and also causes notices to be inserted in the two newspapers published in the city of Hartford, at least ten days before the date of the hearing.

Following the hearing, the Water Bureau determines whether or not public necessity and convenience require the installation of the pipe. If approved, the pipe is laid, and as soon after completion of the work as possible, the cost of the extension is determined from the records of labor and materials used, to which are added overhead expenses, aggregating 23½% (15% on labor, 10% on materials, and 10% on combined labor and material costs, as determined). A summary of the cost of extensions laid on the assessment basis in 1936 is shown in Table 1, and an analysis of the bare cost of laying pipe under various conditions prevailing in 1936 appears in Table 2.

As soon as final costs have been computed, adjusted assessment lists are sent to the Registrar's office, from where the work is billed out and the filing of liens attended to. These liens are permanent encumbrances against the property until the assessments are paid in full, or until the property is foreclosed and sold for non-payment of taxes, in which case lien and assessment are wiped out.

To illustrate the application of the Hartford assessment method, let it be assumed that a section of pipe 500 ft. long is to be laid in a street, filling a gap between the two ends of the street, both of which already have water mains laid in them. If there are no cross streets, the total assessable frontage will equal 1000 ft., and if the cost of the pipe line is $1,500, abutting properties will be assessed $1.50 per foot of frontage.

With this same setup and a 50-ft. cross street without pipe, the assessable frontage is reduced by 100 feet to 900 feet, and the assessment per foot of frontage equals $1,500÷900=$1.667. If there happens to be pipe in the cross street, on one side only, then the two corner lots on that side will be exempt 125 ft. each, a total of 250 ft. for corner lots, plus the 100 ft. for street intersections, giving a net assessable frontage of 1000-250-100=650 ft., and the assessment will be $1,500÷650=$2.307 per foot of frontage. If the pipe in the cross street extends through on both sides, then the two corner lots on the other side will add 250 ft. more of exemption, and the assessable frontage will be reduced to 400 ft., which will result in assessments of $1,500÷400=$3.75 per foot.

TABLE 1—SUMMARY OF MAIN PIPE EXTENSIONS LAID DURING 1936 BY ASSESSMENT METHOD

1	2	3	4	5	6	7	8	9	10	11
SIZE	LENGTH	LABOR COST + 15%		MATERIALS COST + 10%		TOTAL COST + 23.5%		PORTION	PORTION PAID BY	% PAID BY
Inches	Feet	Total	Per Ft.	Total	Per Ft.	Total	Per Ft.	ASSESSED	BUREAU	BUREAU
8	310.6	784.68	2.52½	1,023.53	3.29½	1,989.03	6.40½	793.36	1,195.67	60.2%
12	1075.1	1,662.35	1.54½	3,004.32	2.79½	5,133.34	4.77½	2,664.56	2,468.78	48.2%
Totals	1385.7	2,447.03		4 027.85		7,122.37		3,457.92	3,664.45	51.4%

Costs are given in dollars.
Col. 7 = Col. 3 + Col. 5 plus 10%
Col. 8 = Col. 4 + Col. 6 plus 10%

TABLE 2—ANALYSIS OF PIPE LAYING COSTS — 1936

SIZE	1 ALL HAND LABOR					2 TRENCHING BY MACHINE					3 TRENCHING FURNISHED					AVERAGE 1, 2, AND 3				
	Lgth Ft.	Labor *	Material	Miscell.	Total	Lgth Ft.	Labor *	Material	Miscell.	Tot.	Lgth Ft.	Labor *	Material	Miscel.	Total	Lgth Ft.	Labor *	Material	Miscel.	Total
2"	326	0.91	0.68	0.27¼	1.86¼	..	...	...	...	...	....	.....	.....	....	.....	326	0.91	0.68	0.27¼	1.86¼
3"	..	.....	.....	.....	.....	..	...	...	...	...	329	0.20¾	0.72½	....	0.93¼	329	0.20¾	0.72½	.....	0.93¼
4"	510	0.80½	0.78¼	0.34¾	1.93½	..	...	...	...	...	....	.....	.....	....	.....	510	0.80½	0.78¼	0.34¾	1.93½
6"	1539	1.32	1.15	0.19½	2.66½	1591	0.50	1.06	0.16	1.72	2566	0.23	1.03	....	1.26	5696	0.60½	1.07½	0.09	1.77
8"	6567	1.14	1.52	.....	2.66	4006	1.09	1.32	0.08	2.49	12,844	0.24	1.30	....	1.54	23,417	0.64¾	1.36	.....	2.00¾
10"	540	2.78½	2.28½	0.22	5.39	696	0.76	1.92	...	2.68	....	.....	.....	....	.....	1236	1.64½	2.08¼	0.13	3.85¾
12"	1231	1.52	2.56	.....	4.08	..	...	...	...	...	2369	0.56	2.23	....	2.79	3600	0.89¼	2.34½	.....	3.23¾
†16"	915	0.56¼	2.60	.....	3.16¼	..	...	...	...	...	....	.....	.....	....	.....	915	0.56¼	2.60	.....	3.16¼

† Temporary pipe, laid mostly on surface of ground.
* Labor includes cartage and transportation costs.
All costs are given in dollars.

If the assessment in any of the foregoing examples exceeds the base rate, then the base rate is applied to the assessable frontage, and the difference between the total assessment thus figured and the total cost of the main is borne by the water department.

The examples cited show clearly the marked effect of exemptions on the cost of laying pipe per foot of assessable frontage, as well as on the share of the cost which the water department must assume.

The base rate of assessment has ranged from $1.50 to $1.75 based on 6-in. pipe, and from $1.71 to $1.94 based on 8-in. pipe.

A summary of results obtained under this plan during seventeen years of operations is shown in Table 3. Briefly stated, this table indicates that as of December 31, 1937:

1.	Total cost of work assessed	$1,686,621.35
2.	Total amount assessed	1,284,641.42
3.	Total assessment paid	1,151,877.47
4.	Total interest collected on unpaid balances	72,609.44
5.	Total assessments abated or cancelled	37,408.57
6.	Total active accounts outstanding	82,364.60
7.	Outstanding accounts delinquent, wholly or in part	62,247.08
8.	Total suspense accounts outstanding	12,990.78

From the foregoing, it appears that 76.2% of the total value of work done was billed to abutting property owners, leaving 23.8% to be financed from water department funds. Of the amount billed out (assessed), 89.7% has been paid, 2.9% abated or cancelled, and 1.0% transferred to the suspense accounts, which include all outstanding accounts ten years old or over, and certain other accounts which, for one reason or another, are believed to be most difficult of collection. Interest received on unpaid balances amounted to approximately 4.3% of the total capital investment and is almost double the losses sustained from abatements and cancellations.

The sudden advent of the Depression in 1929 caught the Water Bureau with thousands of dollars tied up in water mains laid in undeveloped real estate tracts, in which all activity soon ceased and from which little or no water revenue was being received. Real estate operators and many other property owners, who had always been prompt in meeting their obligations, hard-pressed from all sides, began to default in their assessment payments, and delinquent accounts increased to a point from which but little recovery has been made. Mortgagees of assessed properties were appealed to, and in many cases accounts were brought to date. Other accounts were turned over to the department's counsel for collection, and in a few cases property was foreclosed and taken over by the department.

TABLE 3—STATUS OF MAIN PIPE ASSESSMENT ACCOUNTS — DEC. 31, 1937.

YEAR BILLED	TOTAL COST OF WORK	AMOUNT ASSESSED		ASSESSMENT ABATED		ASSESSMENT CANCELLED		IN SUSPENSE ACCT.		ASSESSMENT PAID		BALANCE (ACTIVE ACCTS.)		ACTIVE ACCOUNTS DE-LINQUENT
		TOTAL	% TOTAL COST	TOTAL	% TOTAL ASSESS-MENT	TOTAL	% TOTAL ASSESS-MENT	TOTAL	% TOTAL ASSESS-MENT	TOTAL	% TOTAL ASSESS-MENT	TOTAL	% TOTAL ASSESS-MENT	
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
1922	$129,975.40	$ 88,960.68	68.4	$ 114.89	0.1	$ 146.60	0.2	$ 112.26	0.1	$ 88,586.93	99.6			
23	155,891.18	133 879.59	85.9	4,225.35	3.2	100.00	0.1	1,099.28	0.8	128,454.97	95.9			
24	168.934.12	102 560.08	60.7	98.25	0.1	52.50	..	976.99	0.9	101,432,34	98.9			
1925	194,928.37	141,608.92	72.7	25.50	..	670.66	0.5	1,651.74	1.2	139.261.02	98.3			
26	134,644.93	110,444.33	82.0	6.40	..	894.39	0.8	4,716.85	4.3	104,826.69	94.9			
27	119.189.29	94,942.67	79.7	1 645.13	1.7	3,063.43	3.2	.....	..	85,746.49	90.3	$ 4,487.62	4.7	$ 4,487.62
28	154,571.52	104,032.80	67.3	3.603.46	3.5	1,304.96	1.3	29.83	..	92,169.45	88.6	6,925.10	6.7	6,925.10
29	245,483.08	177,584.46	72.3	2.456.43	1.4	3,991.70	2.2	1,822.82	1.0	152.799.19	86.0	16,514.32	9.3	14,483.27
1930	92.166.76	78 298.02	85.0	2 303.56	2.9	3.156.96	4.0	.....	..	63,886.84	80.8	9,550.66	12.2	7,415.02
31	151,747.74	129,051.67	85.0	746.11	0.6	4,079.54	3.2	.....	..	107,875.35	83.6	16,350.67	12.7	12.985.06
32	78,925.94	69,672.58	88.3	1.746.13	2.5	2,976.62	4.3	2,581.02	3.7	49,685.12	71.3	12,683.69	18.2	12,683.69
33	36.737.75	32,761.20	89.1					.....	..	20.887.45	63.8	11,873.75	36.3	2,146.52
34	4,618.03	3,770.69	81.6					.....	..	3,716.92	98.6	53.77	1.4	53.77
1935	6.973.65	6.904.66	99.0					.....	..	6,473.93	93.8	430.73	6.2	.....
36	4,714.15	4,714.15	100.0					.....	..	4.094.00	86.8	620.15	13.2	454.09
37	7,119.44	5,454.92	76.7					.....	..	2,580.78	47.3	2,874.14	52.7	612.92
	$1,686.621.35	$1,284,641.42	76.2	$16 971.21	1.3	$20,437.36*	1.6	$12 990.78	1.0	$1,151,877.47	89.7	$82,364.60	6.4	$62,247.06
										Interest received on unpaid balances 72,609.44				
										$1,224,486.91	95.3			

* Includes:

$12,619.00 on account of foreclosures and Tax Sale of property.
1,820.57 Property foreclosed and taken over by Water Bureau.

$14,439.57

TABLE 4—PERCENTAGE OF TOTAL OUTSTANDING ACCOUNTS DELINQUENT, WHOLLY OR IN PART

DATE	1 TOTAL AMOUNT OUTSTANDING	2 ACCOUNTS DELINQUENT	% 2 TO 1	INCREASE IN AMT. DELINQUENT FOR PERIOD
Feb. 28, 1931	$224,520.97	$36,928.86	16.4	.....
Dec. 31, 1931	205,645.68	42,929.18	20.9	$ 6,000.32
June 30, 1932	220,040.90	49,399.17	22.4	6,469.99
Dec. 31, 1932	212,961.90	63,897.03	30.1	14,497.86
June 30, 1933	229,023.99	75,315.65	32.9	11,418.62
Dec. 31, 1933	219,193.28	86,838.32	39.6	11,522.67
June 30, 1934	194,627.68	89,762.84	46.1	2,924.52
Dec. 31, 1934	184,731.88	101,080.72	54.7	11,317.88
June 30, 1935	167,007.58	103,548.94	62.0	2,468.22
Dec. 31, 1935	153,866.23	103,342.57	67.2	206.37*
June 30, 1936	143,094.45	96,800.46	62.9	6,542.11*
Dec. 31, 1936	123,621.73	90,784.69	73.5	6,015.77*
June 30, 1937	103,470.23	76,603.78	74.1	14,180.91*
Dec. 31, 1937	95,355.38	75,237.84	78.9	1,365.94*

Beginning Dec. 31, 1934 includes Suspense Accounts.
*Decrease.

Table 4 is submitted to show the proportion of total outstanding active and suspense accounts delinquent at six-month intervals, from February 28, 1931 to December 31, 1937. The amounts shown as delinquent are the actual payments in arrears.

In an effort to avoid losses from the depression conditions, a modification of the assessment plan, called the "supply main method," was inaugurated. Under this plan a connection charge (fee), when and if connection is made to the main, is substituted for an assessment laid against abutting property. This charge, originally figured on a minimum frontage of 100 ft., was computed by multiplying the frontage by the base assessment rate for the year in which connection was requested. The minimum 100-ft. frontage ruling was later revised to the actual frontage for lots of less than 100-ft. frontage. Whenever connection is made to a main under these conditions, the property owner is required to submit a drawing showing the exact frontage for which he wishes to provide water service. This information is plotted on a supply main map of the street for office record, and a sketch showing the connection and frontage for which he has paid, is sent to the property owner.

By the installation of these so-called "supply mains," many of which were laid primarily "for the good of the service" in locations and under conditions which would fail of approval under the assessment method, water service has been brought to certain properties that have immediate use for it, without placing the burden of assessment against others who may not need it for years. From the water department's standpoint, the supply main plan insures, assuming full

development of the street, 100 per cent. collection of assessment and elimination of losses through the foreclosure of properties for non-payment of taxes.

To determine the relative merits of the assessment and supply main methods, the results obtained from representative groups of extensions financed under the 10-yr. and 4-yr. assessment plans were compared with results that would have obtained with the supply main method. Results of these studies (shown in Table 5) indicate that the actual cash return on the capital investment in ten years, from the 10-yr. assessment plan, is about 50% greater than under the supply main method, and for the 4-yr. plan, about 72% greater than would have been received in four years under the supply main method. In view of the 17-yr. record of the assessment plan, and its apparent ability to provide a quicker return on the capital investment, it appears to have a definite advantage over the supply main method, in spite of possible defaults and cancellation of assessments.

Nevertheless, under certain property conditions, where as a matter of sanitation a pipe line appears necessary, the supply pipe method offers distinct advantages in its guarantee of payment for value received, in case the property is sold for taxes. Otherwise, there is opportunity for a real estate developer to default and regain possession after clearing the property of the pipe assessment liens.

Conclusions

Briefly stated, our experiences in Hartford with the several methods of financing main pipe that have been outlined indicate that:

1. To finance all main pipe extensions from water revenue would require greatly increased water rates.

2. The guarantee method might be satisfactorily applied and with greater fairness to both consumer and water department, if developed so as to proportion the amount guaranteed over all property actually supplied from a given extension, and if further amended to insure continuity of payment of the guarantee to the water utility throughout the guarantee period.

3. The supply main method eliminates certain risks due to cancellations and foreclosures of properties for non-payment of taxes, which are inherent in the assessment plan. It does not provide as quick return on the capital outlay involved in main pipe extensions as the assessment plan, but permits installations of feeders and lines to strengthen the distribution system in those locations where there is no present nor near future demand for water from the abutting property owners.

While the supply main method is not a satisfactory substitute

TABLE 5—10-YEAR ASSESSMENT EXTENSIONS
ASSESSMENT *vs.* SUPPLY MAIN METHODS

Total Cost	$128,625.35
Total Amount Assessed	98,705.46
Total Assessments Cancelled	545.02
Total Assessments Delinquent	5,587.78

ASSESSMENT							SUPPLY MAIN					
	ANNUAL PAYMENTS									ANNUAL CONNECTION CHARGES		
YEARS	PRINCIPAL	INTEREST	ACCUM. PRINC.	% TOTAL ASSESS-MENT	ACCUM. INT. & PRINCIPAL	% TOTAL ASSESS-MENT	NO. OF CONN.	FRONT-AGE (FT.)	BASE RATE	AMOUNT	ACCUM. AMT.	% TOTAL ASSESS-MENT
1	$34,838.55	$ 63.52	$34,838.55	35.3	$34.902.07	35.4	175	11,266	$1.57	$17,687.27	$17,687.27	17.9
2	35.910.49	423.03	70,749.04	71.7	71,244.59	72.2	148	9,371	1.62½	15,228.40	32,915.67	33.4
3	6,972.15	1,121.96	77,721.19	78.7	79,338.70	80.3	73	4,837	1.60	7,739.36	40,655.03	41.2
4	6,068.85	974.34	83,790.04	84.9	86,381.89	86.4	38	3,020	1.94	5,857.83	46,512.86	47.2
5	3,852.12	291.95	87,642.16	88.8	90,525.96	91.7	42	2,582	1.80	4,647.31	51,160.17	52.0
6	2.339.03	452.76	89,981.19	91.1	93,317.25	94.4	22	1,448	1.78	2,577.67	53,737.84	54.5
7	261.06	98.25	90,242.25	91.4	93,676.56	95.0	11	1,349	1.77	2,387.44	56,125.28	57.0
8	1,148.01	303.05	91,390.26	92.6	95,127.62	96.4	13	722	1.77	1,277.06	57,402.34	58.1
9	478.12	228.30	91,868.38	93.0	95,834.54	97.0	20	1,594	1.71	2,725.47	60,127.81	61.0
10	337.12	137.23	92,205.50	93.4	96,308.89	97.5	32	2,220	1.76	3,906.85	64,034.66	65.0

4-YEAR ASSESSMENT EXTENSIONS

Total Cost	$97,976.33
Total Amount Assessed	73,398.04
Total Assessments Cancelled	1,892.32
Total Assessments Delinquent	14,213.34

1	$13,803.55	$ 63.18	$13,803.55	18.8	$13 866.73	18.9	99	7,016	$1.80	$12,628.80	$12,628.80	17.2
2	11,895.71	383.50	25,699.26	35.0	26.145.94	35.6	19	1,137	1.78	2,023.86	14,652.66	20.0
3	3.770.15	231.99	29,469.41	40.1	30,148.08	41.1	16	935	1.77	1,654.95	16,307.61	22.2
4	9.582.98	1,547.37	39,052.39	53.2	41.278.43	56.2	15	1,827	1.77	3,233.79	19,541.40	26.7
5	4,395.69	679.95	43,448.08	59.2	46,354.07	63.1	27	1,561	1.71	2,669.31	22,210.71	30.3
6	4,552.24	1,099.85	48,000.32	65.4	52,006.16	70.8	38	4,550	1.76	8,008.00	30,218.71	41.2

for the assessment plan as a matter of general policy, it can be used advantageously to supplement the assessment method under the conditions noted.

4. The assessment plan has produced the most satisfactory results of any of the plans described, and is apparently satisfactory to those who desire main pipe extension.

THE NEW SAND WASH BROOK DAM AND RESERVOIR AT PITTSFIELD, MASSACHUSETTS

BY SAMUEL M. ELLSWORTH*

[*Read December 16, 1937.*]

Nearly 25 years ago, Mr. Hiram A. Miller, Consulting Engineer of Boston, presented a paper before This Association entitled "The Additional Water Supply for the City of Pittsfield".† In this paper, he told of the then existing water supply of the city, the need for an additional supply and described the new works which had been recently constructed, including the Farnham Dam, certain collection conduits on the catchment area and certain new supply mains to the city. These works have served the city well until comparatively recently when increased water consumption required that steps be taken to provide greater capacity. With the opportunity to secure financial aid from the Federal government through the P.W.A., the city undertook, and has recently completed, the construction of the Sand Wash Brook Dam and Reservoir, so-called, situated on the upper tributaries of Roaring Brook, above Farnham Reservoir at a site which was considered in Mr. Miller's time as a possible location for a future reservoir.

Existing Supply. While a description of the supply works of the city is given in Mr. Miller's paper, a brief review is necessary here in order to give a reasonably complete picture of the recent additions and the reasons for them. Most of the water supplied to the city comes from surface sources in the hills southeast of the city proper in the towns of Washington, Hinsdale and Dalton. See Fig. 1. Farnham Reservoir, one of the major sources of supply is situated on Mill Brook, nearly 6 miles from the center of Pittsfield. In passing, a word should be said regarding the name of this reservoir, as it is one of the few reservoirs named after an engineer. Arthur B. Farnham, Commissioner of Public Works of Pittsfield and a member of This Association, was engineer in charge of construction of the dam and was thus honored when the city of Pittsfield gave his name to the reservoir. Farnham Dam is a masonry, gravity structure about 500 ft. long and 100 ft. high above the original bed of Mill Brook. The greater part of the catchment area is drained by Roaring Brook whose upper tributaries have been intercepted by conduits built when Farnham Dam was constructed, for the diversion of waters into the

*Consulting Engineer, Boston, Mass.
†This JOURNAL 27, 337 (1913).

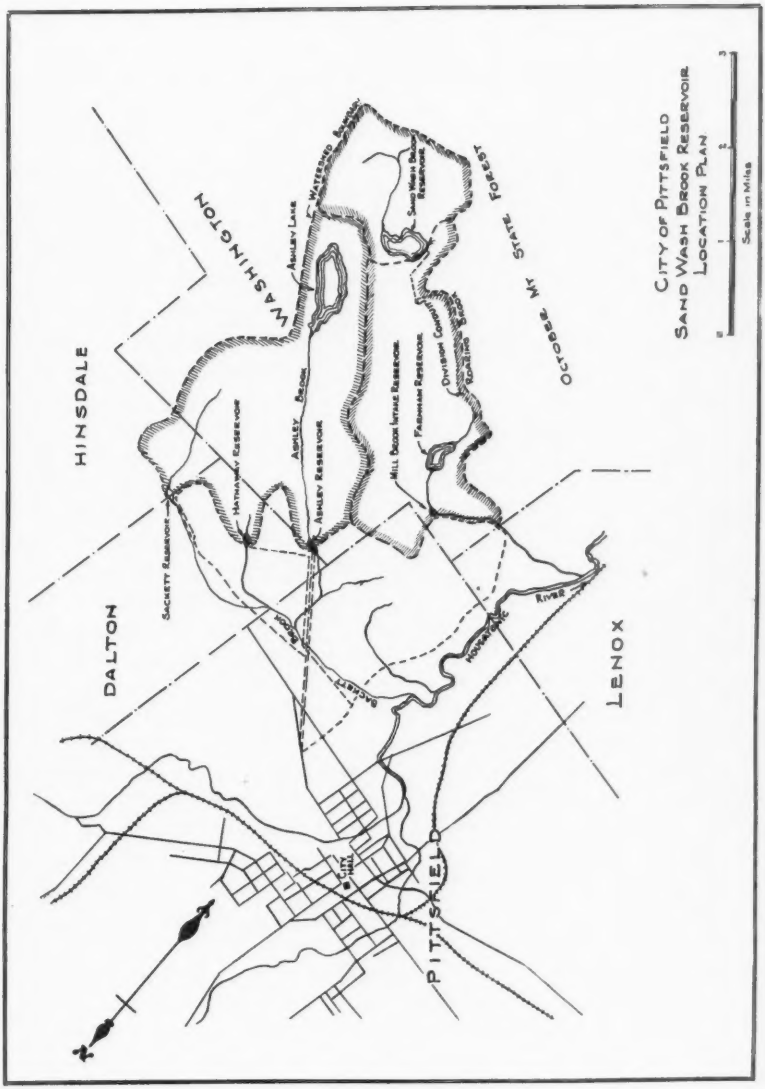
HINSDALE
DALTON
WASHINGTON
LENOX
PITTSFIELD
CITY OF PITTSFIELD
SAND WASH BROOK RESERVOIR
LOCATION PLAN
Scale in Miles
OCTOBER MT STATE FOREST
ASHLEY LAKE
ASHLEY RESERVOIR
HATHAWAY RESERVOIR
SACKETT RESERVOIR
MILL BROOK INTAKE RESERVOIR
FARNHAM RESERVOIR
HOUSATONIC RIVER
CITY HALL

FIGURE 1.

valley of Mill Brook. While the dam is actually on Mill Brook only a small part of the Mill Brook watershed drains to the reservoir. Altogether the catchment area above the reservoir amounts to 3.87 sq. miles.

In addition to Farnham Reservoir, several other sources of supply have been developed in areas adjacent to and north of the Roaring Brook watershed. There are dams on Sacket Brook, Hathaway Brook, Ashley Lake and Ashley Reservoir, as well as an intake dam and reservoir on Mill Brook below Farnham Dam. Ashley Lake is the largest of these sources of supply, its level having been raised by a dam across its natural outlet. The actual storage in Ashley Lake is 450 m.g. but the catchment area is insufficient to support a safe draft of more than 250 m.g. Its outlet discharges into Ashley Brook which in turn flows into Ashley Reservoir which has a capacity of 23 m.g. Altogether the Ashley system has a watershed of 2.47 sq. miles.

In 1913 it was estimated that the safe yield of the old and new supplies was 5.5 m.g.d. The combined watersheds had an area of 10.21 sq. miles, and the combined storage amounted to about 895 m.g. providing 162 days storage on the basis of the estimated yield.

With the gradually increasing rate of water consumption in the city, the capacity of the sources described has been insufficient during periods of low rainfall, and it was necessary on occasions to resort to an emergency supply pumped from Onota Lake situated on the opposite side of the city. This supply had been used in much the same manner prior to the construction of Farnham Reservoir. It is not an acceptable supply from the standpoint of quality since it is subject to pollution and thus has been used only in emergencies.

Additional Supply. Several years ago, when the city realized that the limited yield of its upland water sources would have to be supplemented, certain studies were undertaken looking toward the development of additional sources. The new sources considered at that time involved a comparatively large expenditure of money although their development would have made available a very substantial increase over the yield of the existing sources. However, in view of the fact that the population of Pittsfield has shown only a moderate increase during the past 15 years, and considering the pressing need during the recent depression for suitable projects for unemployment relief, it was decided to proceed with a project that could be constructed at moderate cost by further developing the existing sources of supply. By the construction of an earth dam on Sand Wash Brook, at the site considered in 1913, additional storage of about 265 m.g. could be secured, and the safe yield of the existing sources

could thereby be increased by about 700,000 g.p.d. or sufficient to obviate the necessity of using the emergency supply from Onota Lake. The project was in many respects an ideal one for unemployment relief, involving the use of a large proportion of unskilled labor and local materials, and in operations such as clearing and grubbing labor was the principal item of cost.

Sand Wash Brook Dam. The dam itself is about 1600 ft. long with a maximum height above the floor of the valley of a little more than 40 ft. The site of the dam and the sides of the reservoir basin are underlaid with ledge rock, covered for the most part by a thin overburden of loose top-soil or occasionally by pockets of hard-pan. There are numerous outcroppings of ledge rock, and in many places the rock is jointed and fractured and large blocks at the surface have been loosened by frost action. The dam crosses two gorges, one through which passes Sand Wash Brook and the other called the South Gorge, where core borings indicated that the loose and broken rock extended from 15 to 20 ft. below the ground surface. For most of the length of the dam, however, the rock surface was fairly sound and required little or no treatment.

With ledge rock so close to the surface, consideration was given to the construction of a concrete masonry dam. Comparative cost estimates, however, showed that chiefly because of the lack of suitable materials for concrete aggregate within a reasonable hauling distance of the site, an earthfill dam would be decidedly more economical.

The only material available in sufficient quantity for the earth embankment consisted of a very dense hard-pan containing a large percentage of fine clay. This could be obtained from a borrow area less than ¼ of a mile from the dam. The quality of material was found to be quite uniform over the entire borrow area. Typical mechanical analyses of samples of material, which were made under the direction of Dr. Arthur Casagrande of Harvard University, are shown in Fig. 2. The curves show that 60% of the material was finer than 0.2 and 0.08 mm, respectively, and that 10% was finer than 0.001 mm, giving uniformity coefficients of 200 and 80, respectively. The maximum density was found to occur with a water content of 13 to 14% when the unit weight amounted to about 136 lb. per cu. ft. The variation between water content and the unit weight of material is shown on Fig. 2. The coefficient of permeability varied between 0.006×10^{-4} cm. per sec. and 0.118×10^{-4} cm. per sec. indicating a very impervious material. It was evident that by giving careful supervision to the spreading and rolling operations and by maintaining a water content reasonably close to the optimum, seepage through this material would be quite low. While it was ideal for

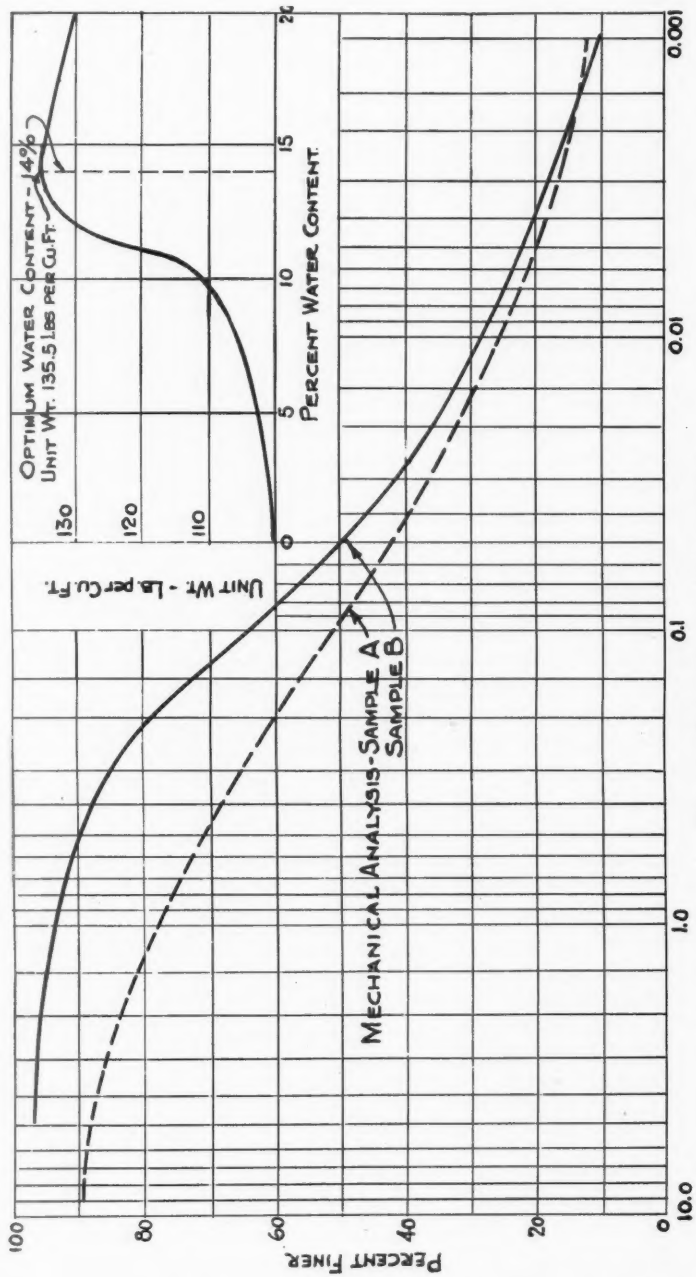

FIG. 2—SAND WASH BROOK DAM—MECHANICAL ANALYSES OF EMBANKMENT MATERIAL AND OPTIMUM WATER CONTENT.

securing an effective cut-off against seepage, its use for the dam as a whole would require suitable drainage to insure the stability of the downstream slope. On the basis of a downstream slope of 1 on 2 and a top width of 20 ft., the seepage losses were estimated to lie between 0.02 and 0.1 gal. per foot per day with a height of 40 ft. On the basis of the same dimensions, the line of seepage would intercept the downstream face at about one half way between toe and crest, thus requiring the use of pervious, draining material over the downstream face of the dam or suitable drainage beneath the embankment so as to bring the seepage line below the toe of the slope.

While the report of the analyses indicated that a stable and impervious dam could be constructed without the use of a concrete core-wall, there were several factors that led to the adoption of such a core-wall in the final design. With a material as impervious as that available for the embankment and with ledge rock at or close to the ground surface over most of the dam site, it was necessary that a considerably quantity of suitable pervious material be secured for the drainage of the embankment. Such material could not be obtained within a reasonable hauling distance of the dam site. Furthermore, it was apparent that the embankment material would require great care in placing in order to secure uniformity of compaction and prevent localized seepage. The size of the project did not seem to warrant extensive field laboratory control for the placing operations, and without such control it was felt that too much reliance must be placed upon the judgment of the engineers in the field. Inasmuch as the downstream slope of the dam could be steeper with a concrete core-wall than without, and because the quantity of relatively expensive pervious material required with such construction would be much less than otherwise, the saving in cost by omitting the concrete core-wall became insignificant when compared with the greater security offered by such construction.

A typical cross section of the dam is shown on Fig. 3. The top of the embankment is at elevation 1901. From this elevation to elevation 1881 the slope of the upstream face of the embankment is 1 on 2½ and below elevation 1881 it is flattened to 1 on 3¼. The downstream face has a slope of 1 on 2, a berm 3 feet wide being provided at elevation 1881. The core-wall footing is on sound ledge rock throughout the entire length of the dam. Where the rock had been exposed to frost action, particularly at the sides and bottoms of the two gorges it was so jointed and fractured that a considerable quantity had to be removed in order to obtain a suitable foundation for the core-wall footing. Grout holes were placed at frequent intervals along the core-wall trench but the quantity of cement grout

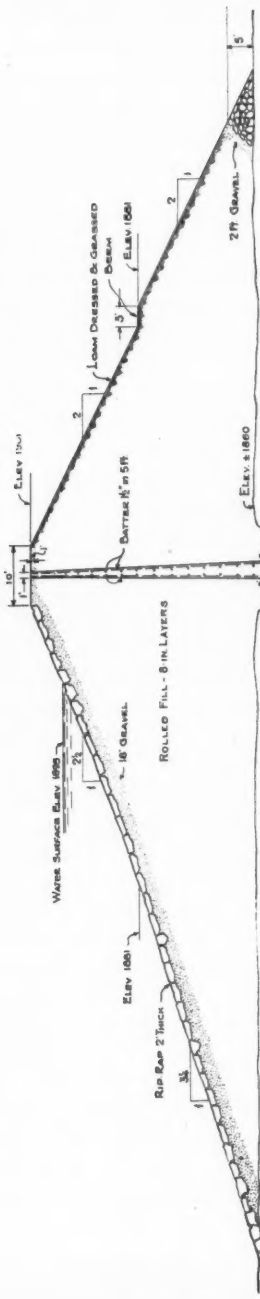

FIG. 3—CROSS SECTION OF SAND WASH BROOK DAM.

actually used was quite small except where the dam crossed Sand Wash Brook and the so-called South Gorge. It will be seen from Fig. 3 that 2 ft. of riprap underlaid by 18 in. of gravel were provided on the upstream face of the embankment. The downstream toe consisted of a rock-fill backed up by a layer of graded gravel 2 ft. thick to provide a blanket between the fine material and the rock-fill toe. The downstream face of the embankment was dressed with 12 in. of top-soil.

Considering the general character of the material for the embankment and the fact that it would come from the borrow area in a lumpy condition, the specifications required that it be placed on the embankment in layers not exceeding 8 in. in thickness before rolling, and that the rolling should be done by a sheepsfoot, tamper-type roller drawn by a caterpillar tractor. Each layer of the rolled fill was required to be gone over a sufficient number of times to break up all lumps and secure the desired degree of compaction. It was required, as is usually the case, that the surface of each layer of material should be sprinkled before spreading another layer. Actually the material was such as to require but little sprinkling to obtain the necessary compaction. In other words, the water content, as the material came from the borrow area, was so close to the optimum that relatively little water needed to be added. As is customary in dams with thin concrete core-walls, the fill on both sides was kept as near the same elevation as practicable, and to prevent the formation of water pockets, the surface of the fill was maintained with sufficient slope to provide drainage away from the core-wall during the placing operations.

During the placing of the embankment a relatively simple field test was used to control the water content and to give a measure of the consolidation of the material. Cores of undisturbed material from the borrow area and compacted material from the dam embankment were taken by a simple coring device made from a piece of 6-in. steel pipe, about 6 in. long. A U-shaped handle was welded to one end of the pipe while the other end was bevelled to form a cutting edge. In collecting a sample the pipe was forced down into the material to be sampled to a little more than its full length. By means of a shovel the lower end of the core was cut away from the surrounding material. Both ends of the core were then trimmed off even with the ends of the sampler. By weighing the sample the unit weight of the material was obtained and the data used as a guide in maintaining the optimum water content. The unit weights of embankment samples determined in this manner generally varied from 132 to 138 lb. per cu. ft.

The design called for a core-wall of reinforced concrete having a thickness at the top of 12 in. and a batter on each side of 3 in. in 10 ft.

Vertical expansion and contraction joints were placed at intervals of 30 ft. and consisted of 20-oz. sheet copper folded in the middle around a strip of sheet mastic so as to form a watertight cut-off at each joint.

Spillway. The spillway is situated near the center of the dam on the north shoulder of the divide between the valley of Sand Wash Brook and the South Gorge. At this point the top of the dam is only a few feet above the original ground surface. The surface of the ledge rock here rises to within 4 to 10 ft. of the flow line of the reservoir, thus giving an ideal foundation for a low concrete masonry spillway. The latter is 90 ft. long between the wing walls, which are of concrete faced with stone masonry. The spillway has a designed capacity of 1280 cu. ft. per sec. equal to 765 cu. ft. per sec. per sq. mile, based upon a head on the crest of 2.5 ft. The top of the dam is 6 ft. above the spillway crest giving a freeboard at the estimated maximum discharge of 3.5 feet. The discharge from the spillway is carried through an open channel, partly paved and partly cut through ledge rock, to the original bed of Sand Wash Brook.

Gate House. The gate house is located within the embankment, its upstream face being in the same plane with the core-wall. The outlet pipes from the reservoir enter the bottom of the gate chamber and are arranged to permit drawing water from two levels. These pipes are of cast iron, 18 and 24 in. in diameter, respectively, the combined capacity being sufficient to provide for the flow of Sand Wash Brook during construction. The 24-in. pipe is connected to a low-water intake near the upstream toe of the dam while the 18-in. pipe turns up into an intake chimney drawing water down to a depth of 20 ft. below the spillway which is at elevation 1895. Downstream from the gate house a concrete pipe and diversion conduit is provided. During construction this conduit was used for the diversion of water. An 18-in. discharge pipe from the gate house is carried through this conduit. With the completion of the work on the dam embankment, the upper end of the diversion conduit was sealed by a concrete plug. The gate house superstructure is of quarry-faced stone masonry, with interior walls of face brick.

Aerator. The discharge pipe is carried below the dam to an aerator, consisting of a circular basin with an outside diameter of 42 ft., with aerating nozzles clustered about the center. There is one 4-in. nozzle at the center, an intermediate ring of eight 3-in. nozzles and an outside ring of sixteen 2½-in. nozzles. The inlet piping to the

aerator nozzles is arranged so as to permit using the nozzles in various combinations depending upon the rate at which water is being drawn from the reservoir and the head available. Water from the aerator discharges into an outlet chamber and over a measuring weir into a paved open channel and thence to the original collection channels that were constructed for the diversion of water into Farnham Reservoir in 1913. In the original project it was not expected that provision for aeration would be made as it was anticipated that the basin of Sand Wash Brook Reservoir would be stripped as was done 25 years before in the case of Farnham Reservoir. When the time came to prepare final plans, however, it was decided to omit the stripping of the reservoir basin, and the aerator was provided in the hope that it would compensate in part for the elimination of the stripping program.

The quality of Roaring Brook is affected by a large swampy area above Sand Wash Brook Reservoir. The color of Roaring Brook, just above Farnham Reservoir, in 1936 averaged 55 p.p.m. which was reduced by storage in Farnham Reservoir to 31 p.p.m. In an attempt to improve the quality of the water a system of drainage ditches was constructed during 1937 as a W.P.A. project through and along the margins of the swamp above the Sand Wash Brook Reservoir.

Cost of Construction. The construction of the dam and reservoir was carried out under two contracts. Contract No. 1, the general contract, was awarded to the Lindholm Construction Company of Pittsfield, and included the construction of the dam, gatehouse and spillway, the clearing and grubbing of the reservoir basin, the building of about 2 miles of gravel road into the dam site, and the construction of the spillway channel. A comparison of the unit price bids for Contract 1 is shown at the end of this paper. The final cost of the work under Contract No. 1 amounted to $210,000. Contract No. 2 was awarded to Simpson Brothers Corporation, of Boston, and covered the construction of the aerator, a concrete bridge and culvert, and improvements to the outlet conduits below the dam. The cost of the work under this contract amounted to $40,700. Including the aerator nozzles, which were purchased under a separate contract, the total cost under all contracts amounted to $250,902.41. While the final grant from the P.W.A. has not as yet been paid to the city, it is expected that the net cost to the city will be close to $158,000. Thus, the cost of the new supply to the city based upon the increase in the safe yield of these sources will amount to $226,000 per m.g.d. of increased yield.

The design and construction of the works was carried out under

the direction of Mr. Arthur B. Farnham, Commissioner of Public Works. The author acted as consulting engineer on the preparation of contract plans and specifications and on the construction of the project. Professional advice on special features of the project was furnished by the author's associate, Mr. Howard M. Turner, Consulting Engineer, Boston; Dr. Charles P. Berkey, geologist, New York City; and Dr. Arthur Casagrande, Assistant Professor of Soil Mechanics at Harvard University.

COMPARISON OF UNIT PRICE BIDS ON THE GENERAL CONTRACT—CONTRACT 1

	ESTIMATED QUANTITY	UNIT PRICES LOW* BIDDER	SECOND LOWEST	HIGH BIDDER
Clearing only	60 acres	$100	$150	$150
Clearing and grubbing	50 acres	240	300	400
Grubbing only	10 acres	200	200	250
Earth Excavation				
Dam and structures	13,500 cu. yd.	1.00	1.00	1.50
Roads	8,000 cu. yd.	0.80	0.60	1.00
Soil in reservoir	700 cu. yd.	3.00	3.00	1.50
Rock Excavation				
In preparation of surfaces for concrete	1,000 cu. yd.	4.00	7.00	6.00
General	200 cu. yd.	2.00	3.50	5.00
Drilling holes in rock	160 lin. ft.	2.00	1.00	2.00
Grouting				
Apparatus and equipment	Lump sum	500.00	1,500.00	1,000.00
Cement for grout	80 bags	2.00	1.00	1.00
Grouting exclusive of cement	15 cu. yd.	5.00	50.00	30.00
Refill and earth embankment	51,000 cu. yd.	0.80	1.00	0.50
Gravel—dam embankment	5,000 cu. yd.	1.80	1.25	0.80
Rip-rap	7,000 cu. yd.	2.00	2.50	3.00
Surface dressing and grassing	2,100 cu. yd.	2.00	1.50	2.0
Gravel surface for roadway	11,500 sq. yd.	0.48	0.30	0.70
Rock fill for road and drainage ditches	1,700 cu. yd.	2.00	2.50	3.00
Concrete masonry	3,000 cu. yd.	17.00	14.00	15.00
Rubbing concrete	1,650 sq. ft.	0.10	0.10	0.10
Steel reinforcement	199,000 lb.	0.05	0.04	0.05
Copper expansion joints	500 lin. ft.	1.00	1.00	0.40
Stone facing and coping	30 cu. yd.	50.00	20.00	50.00
Dressing stone masonry	250 sq. ft.	1.00	1.00	2.00
Miscellaneous iron and steel	2,500 lb.	.15	.15	.20
Cast iron pipe				
B. & S. incl. fittings	115,000 lb.	0.05	0.045	0.05
Flanged incl. fittings	5,500 lb.	0.10	0.10	0.08
Metal Culvert Pipe—12 in.	80 lin. ft.	2.00	1.75	3.00
18 in.	20 lin. ft.	3.00	2.50	5.00
Gate Valves, Sluice Gates with Extension Stems and Stands				
18-in. valve	1 each	300.00	400.00	400.00
18 in. valve	1 each	310.00	425.00	500.00
24-in. valve	1 each	500.00	650.00	700.00
18-in. sluice gate	1 each	350.00	400.00	450.00
24-in. sluice gate	1 each	400.00	450.00	500.00
Gate house superstructure	Lump sum	2,350.00	4,000.00	2,500.00
Lumber left in place	5 M. ft. B. M.	40.00	100.00	40.00

* Awarded contract.

EXPERIENCES WITH REINFORCED CONCRETE STANDPIPES

BY CHARLES W. SHERMAN*

[*Read March 17, 1938*]

It has been said that we learn from our failures quite as much as from our successes. Reinforced concrete standpipes have not generally proved to be economical or satisfactory structures, and therefore they may be classified as failures, even though no case of structural collapse is known to have occurred—yet. The appearance of some of them is such, however, that it is not easy to see why they do not fall down, unless they are sustained by the "force of habit".

Early in the present century, when the use of reinforced concrete was becoming relatively common, it appeared to some designing engineers and contractors that water works standpipes might be built of this material. At that time the cost for reinforced concrete standpipes was somewhat less than for those built of steel; neither engineers nor contractors anticipated any serious difficulty in securing water-tight construction, and water commissioners believed that a one-year guaranty by the contractor would ensure a permanently tight structure; and it was expected that there would be no cost for maintenance, and no inconvenience corresponding to that caused by the periodic painting of a steel standpipe. As one water commissioner to put the case:[1]

> "The advantages of a concrete-steel standpipe over a steel standpipe are:
> First. The cost would be $..... less.
> Second. No cost of maintenance.
> Third. There seems to be no limit to the life of such a structure."

The supposed advantages of a reinforced concrete standpipe have been expressed in another way by a superintendent who, at the time of writing, had lived for 5 years with such a structure. He said:[2]

> "Quite naturally we are attracted to the picture of a beautiful white standpipe, one that never grows old in the sense of deterioration. The longer it stands there, the more solid and enduring it becomes. It will never require painting, and consequently will not have to be emptied; always full, always ready.
>
> "The picture is ideal. Its attainment, I regret to say, is not always accomplished. . . .
>
> "A water-works standpipe is not generally considered a thing of beauty. On the contrary they are often spoken of as a blot on the landscape, a neces-

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[1]This Journal, **20**, 305 (1906).

[2]This Journal, **29**, 170 (1915).

sary evil that, in the case of an iron structure, depending upon the individual tastes of those happening to be in authority, may be a dark red, a dreary black, or a battleship gray.

"On the other hand, a leaky concrete standpipe is certainly no improvement, and when in addition to the unsightly appearance is added the feeling of insecurity, which leakage in a structure of this character naturally creates, there is not much that can be said in its favor."

During the period 1905-1915, a considerable number of reinforced concrete standpipes were built. Since the latter date there have been few new structures of this type, but experience in their maintenance and repair has been plentiful. However, it is not easy to obtain much information about this experience. This paper presents a summary of such information as is readily available, or which could be obtained by personal inspection of such structures of this type as could be easily reached.

Perhaps it should be stated that these notes relate solely to reinforced concrete structures, in which steel is bonded into the concrete, and not to tanks in which the steel is prestressed and precautions are taken to avoid a bond between the concrete and the steel.

In 1915 This Association devoted a meeting to the general subject of concrete standpipes. Papers describing the standpipes at Attleborough, Mass., and at Westerly, R. I., were read, followed by a general discussion in which considerable information relative to the construction and maintenance of many such structures was presented. Reference is made to the JOURNAL of This Association, 1915, pp. 169 to 201. Photographs showing the condition of several standpipes at that time were included. Some of them showed that serious damage had already been suffered; others showed stains that made the structures unsightly; and still others showed little or no indications of deterioration.

The following notes relate to the expriences with and present condition of 12 reinforced concrete standpipes which the author has been able to inspect or about which he was able to obtain information. They are arranged in the order in which they were listed in the table on p. 186 of the JOURNAL for 1915.

Attleborough, Mass.[3] This standpipe is 50 ft. in diameter and 102 ft. high. The original cost of the standpipe in 1905 was $34,000, and the cost of the membrane in 1914 was $3,000.[4] Present condition is indicated by the following quotation from the 1935 report of the department of public works: "This department has times without number recommended that improvements be carried out at

[3]See This Journal, **29**, 179 (1915), which describes the various attempts to attain tightness by plastering, Sylvester process wash, and finally (1914) a felt and asphalt membrane, together with 8-in. brick wall and 2-in. air space *outside* the standpipe.

[4]Cost of brick enclosure was not stated.

the standpipe location in order to protect the water supply of our public. Once again we urge immediate action in this respect."

The annual report for 1915 stated that there had been no evidence of leakage since the membrane was applied in 1914. The reports for 1916 to 1920 state that the standpipe was in satisfactory condition. It appears probable that the membrane failed sometime between 1921 and 1925.

Waltham, Mass.[5] The standpipe is 100 ft. in diameter and 43 ft. high. Attempts at waterproofing in 1913 and in 1915-17 were unsuccessful. About $1,000 was spent on these attempts. In 1922 a membrane of felt and asphalt was applied to the interior surface, and protected by a 4-in. brick wall, the ½-in. space between the membrane and the wall being filled with cement mortar. The original cost of the standpipe was about $26,000 and the cost of waterproofing in 1922 was about $16,500.

After 15 years the standpipe appears to be tight, but its appearance as shown in Figure 1 is bad. Disintegration of the exterior

FIG. 1—REINFORCED CONCRETE STANDPIPE, WALTHAM, MASS., 1937.

surface has continued, probably as a result of frost action. There are some 15 or 20 places where plastering of the exterior surface is needed,—spots ranging from about 10 to about 200 sq. ft. in area. In some places the spalling extends to a depth of 2 or 3 inches. The cornice and parapet also show signs of disintegrating.

Manchester, Mass. The diameter of this standpipe is 50 ft.,

[5]See THIS JOURNAL **21**, 325 (1907), and **38**, 297 (1924).

and its height 72 ft. It was built in 1908. "The usual troubles" developed and in 1915 the standpipe was very unsightly.[6] Considerable money was spent on repairs, but no permanent improvement was accomplished until 1919, when a felt and asphalt membrane, protected by a 12-in. brick wall was provided on the inside. The cost of this waterproofing was about $12,500. It is believed that there has been no leakage since the membrane was provided.

Westerly, R. I. Built in 1910, this standpipe is 40 ft. in diameter and 70 ft. high. Its original cost was about $18,700.

"At the time of the construction of the concrete standpipe in 1911 it was found necessary to waterproof the walls of the pipe, and this was done by applying a membrane of a specially prepared Neponset compound and five layers of waterproof felt on the interior walls. The application of this membrane secured the desired result at that time and the standpipe was tight. Within the last two years the pipe has leaked at the joints, and the leakage increased to such an extent it was feared that the frost would damage the concrete and cause it to spall off exposing the steel reinforcing.

"The water was drawn off on November 4, 1926, and an examination of the interior of the pipe showed that the waterproofing membrane had separated from the wall for a distance of 15 or 20 ft. down from the top, and over the remainder of the pipe the membrane was covered with large blisters, showing deterioration of the material. On the bottom of the pipe, where the waterproofing was covered with concrete, the waterproofing material was in good condition. As the life of the material was gone, the work of cleaning the surface of the concrete and removing all of the old waterproofing material was commenced at once.

"In the meantime, while this work was going on, a study was made of the problem and expert advice obtained for the best method of waterproofing the structure. After a thorough investigation it was decided to again apply the membrane method and protect the waterproof membrane from the action of the air and water with a four-inch brick lining. . . .

"Immediately following the application of the waterproof membrane, a four-inch brick core was constructed from the floor to the top of the wall. A one-half inch annular space between the outside waterproof course and the inside face of the brick work was carefully filled with mortar. After completion of the brick work the walls were washed down with a Cement and Celite wash. The floor was then covered with Compound and Felt and three inches of Cement mortar. . . .

"The total cost of the work was $11,158.81. . . .

"As the original unprotected waterproofing material lasted about fourteen years, we feel confident that, with the brick lining to protect the waterproofing, and the careful painstaking work of everyone connected with the project, the standpipe will remain tight for a much longer period."[7]

Rockland, Mass. (Rockland and Abington Water Supply). This standpipe has a diameter of 46 ft. and a height of 104 ft. It was built in 1910. The Report of Rockland Water Commissioners for

[6] See This Journal, 29, 184 (1915), Plate 9.
[7] Quoted from Westerly Water Department Report, 1927.

1931 includes a report of Weston and Sampson on the standpipe. According to this report an attempt at waterproofing was made in 1921, by applying a coat of asphalt to the interior surface. This was not successful in stopping leakage. Some repairs were also made to the exterior surface by plastering. These also were of only temporary value. Inspection in summer of 1937 (exterior only) showed the structure in very bad condition, the worst places being between 10 and 60 ft. above the ground. As shown in Figure 2, large areas

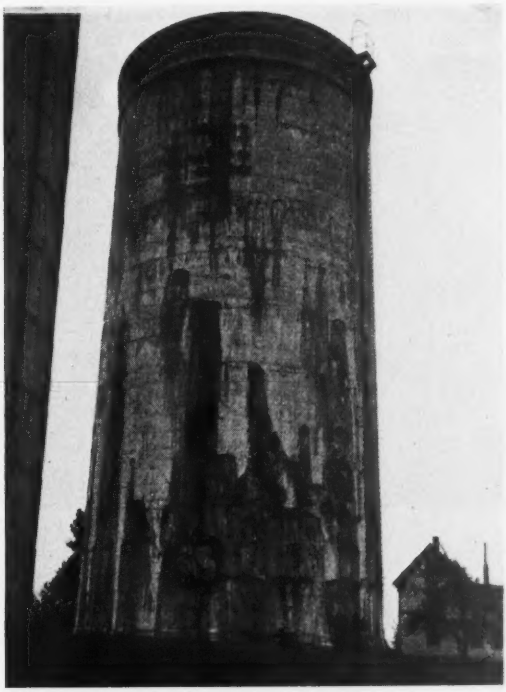

FIG. 2—REINFORCED CONCRETE STANDPIPE, ROCKLAND, MASS., 1937.

of exterior concrete have spalled off, some of them apparently to a depth of 6 in. or more. Considerable steel is exposed, but it does not appear to have been damaged by rust; however, in one or two exposed places, the lap of the steel bars is inadequate and the concrete has spalled off so that no tensile stress can be transmitted through the steel at these places. Slight leakage is apparent at many points and there is one vertical strip about 5 ft. wide and probably

50 ft. high, where a film of water of appreciable thickness flows to the ground. No spurting leaks were seen.

Brockton, Mass. The "Twin Reservoirs" in Avon, each 160 ft. in diameter and 26.5 ft. high, were built in 1911. Owing to the comparatively small depth of water in these tanks, it is unlikely that anyone thought that they would ever require an expenditure for repairs, or that seepage through the concrete would be of consequence. An early photograph[8] indicates that the structures were in perfect condition, although the joints between successive pourings were distinctly visible. The present condition is different, as shown

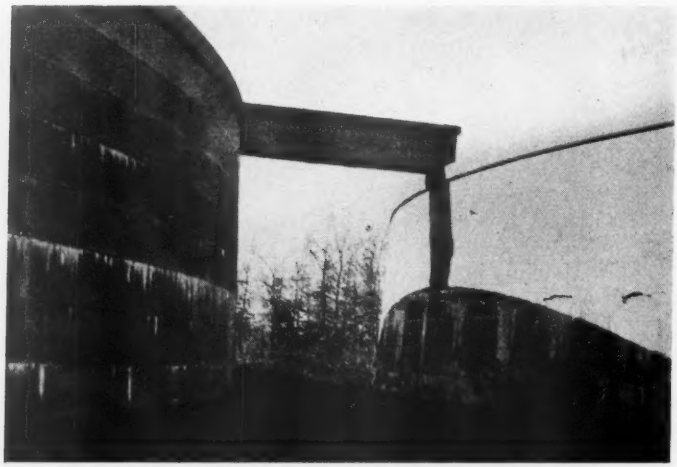

FIG. 3—TWIN RESERVOIRS, BROCKTON, MASS., 1937.

in Figure 3 and by the following quotation from the Water Commissioners' Report for the year 1936:

"Once again we recommend for the very near future the waterproofing of the Twin Reservoirs in Avon. These reservoirs were constructed in the year 1911, and up to date there has never been one cent spent in repairs. They consist of two reinforced cement tanks 160 ft. in diameter, and 26½ ft. high, containing 4 m.g. each.

"For the last few years there has been a seepage from the inside combined with the New England climatic conditions on the outside that has caused a spalling on the outside surface, and although there are no leaks large enough to reach the ground, the outside surface is deteriorating and spalling much faster each year, and as these tanks constitute our total holding or reserve supply, we consider it of major importance that the waterproofing of these tanks be considered as an important project from the standpoint of safety and economy, to

[8]This Journal, **29**, 200 (1915), Plate 19.

be attended to in the very near future. And the money necessary to carry out these repairs should be provided by the City Government immediately."

Careful examination of the interior condition of these reservoirs when they were cleaned about a year ago shows that the concrete has suffered no damage on the interior surface. It is also worthy of note that the wall above the ordinary water level shows no indications of damage, either on the inside or outside.

Spalling of the exterior surface is much more considerable on the north reservoir than on the south reservoir. The maximum depth to which this spalling is extended probably does not exceed 2½ in.

No evidences of seepage through the concrete walls were discovered until two years after the original construction, at which time a small amount became visible at the fifth joint above the ground level of the south reservoir. The north reservoir showed no indications of seepage until 1928, but in that year white streaks from the efflorescence became noticeable and spalling of the concrete began on the north reservoir. Since that time the efflorscence has become decidedly marked on the south reservoir but spalling has been insignificant. The greatest damage from frost action has been on the north reservoir.

The total cost of these two reservoirs in 1911 was $83,915.

Weston, Mass. (Paine's Hill Reservoir). This structure, 50 ft. in diameter and 38 ft. high, is a composite one, the lower half being a rubble masonry reservoir built in 1896 and embanked with earth nearly to the top of the masonry. In 1911, the stone walls were lined with 4 to 6 in. of reinforced concrete, and a concrete shell similar to that of a standpipe was built, extending about 20 ft. above the earth embankment.

Nothing is known about the condition of the underground structure, except that no indications of leakage have appeared at the surface of the ground. The concrete structure above ground has leaked considerably and an appreciable area of exterior surface has spalled off. Several years ago an attempt was made to stop leakage by applying a material to the inside surface which was expected to fill the pores of the concrete, and the outside was repaired by plastering. At the present time the leakage is not serious, and the outside is in fair condition. However, within a few years it will be necessary to make extensive repairs, or to abandon the reservoir and replace it by a new structure. See Figures 4 and 5.

Lexington, Mass. The diameter of this standpipe is 30 ft. and its height 104.5 ft. It was built in 1912. See Figure 6. The following extracts from the annual town reports give an accurate picture of the experience with this structure.

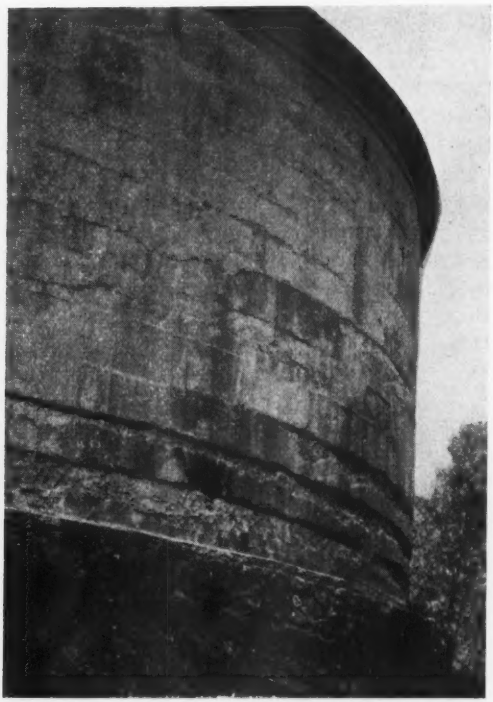

FIG. 4—REPAIRS BEING MADE ON WESTON, MASS., RESERVOIR, ABOUT 1932.

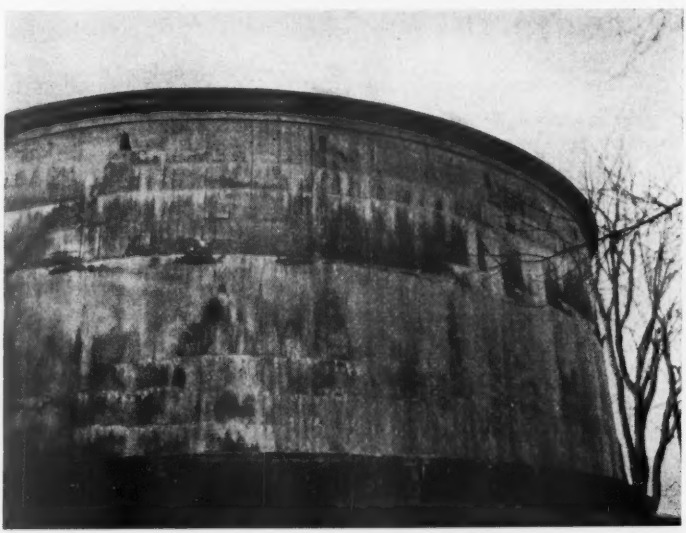

FIG. 5—WESTON, MASS., RESERVOIR, 1937.

FIG. 6—REINFORCED CONCRETE STANDPIPE, LEXINGTON, MASS., 1937. STANDING EMPTY.

1915 "The concrete standpipe has required considerable of the Board's attention, due to an excessive amount of seepage through the concrete walls. This condition obtains more or less in all standpipes built of concrete and is particularly noticeable in standpipes as high as this. The actual amount of water that escapes, however, is extremely slight. Your Commissioners have consulted the best expert advice obtainable in their endeavor to overcome this trouble. They found that with inside patching and stopping slight leaks the conditions could be temporarily improved, but the standpipe could not be made permanently tight without going to a large item of expense which in the judgment of the Board was unwarranted at this time.

"The Commissioners employed as consulting engineers the firm of Messrs. J. R. Worcester Co., who, with the coöperation of the contractors who built the standpipe, made repairs to the most serious spots where the concrete had become dislodged. This should postpone any heavy expenditure until another winter has passed, thus allowing sufficient time to elapse to permit all weak spots to appear. Then the Board will undoubtedly recommend to the Town that steps be taken to put the standpipe in permanently tight condition."

1919 "The Board have called in consultation Messrs. J. R. Worcester & Company, Engineers of National reputation, who after a study of the problem have made recommendations and estimates based on tentative bids. To make

the tank permanently tight and of uniform and good appearance on the outside will require the expenditure of approximately $27,000. There is a possibility, however, of rendering it tight and serviceable for about $17,000."

1920 "The repairs to the standpipe have received much attention from the Commissioners. That we might have the benefit of all available information, a meeting was held in May, 1920, at which were present those who had served on the Board in previous years, Mr. Simpson of Simpson Brothers (the Contractors), and representative of J. R. Worcester Company, our Consulting Engineers. The construction of the Tank and its defects were discussed at length and in the frankest manner, and it was clearly demonstrated that the standpipe was built according to specifications and in workmanlike manner, and further, that the specifications were drawn with all engineering skill and knowledge available at that time. Standpipes built at the same time in other localities have been equally disappointing, for the technical knowledge of concrete in 1912 was far from being as complete as in 1920.

"Following the survey and report of Messrs. J. R. Worcester & Company, an appropriation of $14,000 was granted to carry out the necessary work on the interior. On the recommendation of the Engineers, the Board was unanimous in its decision to adopt the 'Ironite' method of water proofing. This method consists of removing all loose material and cutting out all seams, then spreading on the Ironite Compound to seal all voids and make a new unbroken surface. While this method is somewhat newer than others, it seemed to best meet the requirements of waterproofing a porous concrete tank 105 ft. high exposed to considerable fluctuations in temperature, ice, water pressure and wind pressure. Furthermore, the waterproofing compound must be insoluble, tasteless and easy to repair. That we might have further information, our Superintendent visited Willard, Ohio, to examine a large concrete water tower which had been waterproofed by the Ironite method some time previously.

"The Ironite method also proved much less expensive than other methods considered, the contract price for the interior work being $7,850.00 No bond was taken but in lieu thereof, $4,000.00 of the contract price was withheld, and, provided the standpipe remains waterproof and water-tight, $800.00 is paid each year for a term of five years. On these deferred payments the Town is obligated to pay interest at 5%, but the $4,000.00 has been deposited in a special account at the Lexington Trust Company bearing 5% interest for the five-year period so that the Town is under no expense whatever for this interest item.

"Messrs. J. R. Worcester & Company considered it very essential that all loose concrete should be immediately removed from the exterior of the standpipe until a permanent hard surface was exposed. By vote of the Town, the balance of the appropriation was made available for this work and the loose concrete was all removed and a coating of Ironite applied before very cold weather set in. It was found necessary to remove a far greater proportion of the outside material than was at first apparent and when the appropriation was exhausted, by vote of the Town, $2,000.00 surplus funds of the Water Department were transferred to the standpipe account. In addition, the further sum of $1,436.44 will be advanced by the Water Department from current funds, thus making the total cost of the repair work to date $17,436.44. The exterior is now ready to be covered with concrete and additional reinforcement, the specifications for which are already in our hands. We are informed by our Engineers that a fair replacement value of the Standpipe after completion of the work on the exterior will be $45,000.00. An appropriation of $15,000 will be required in 1921 to complete the exterior work.

1921 "The work on the standpipe has progressed during the greater part of the Fall and early Summer. The work was carried out under the engineering direction of the J. R. Worcester Company who drew up specifications for the work. Early bids, in the opinion of the Board, were too high but we were at last successful in securing a bid of $11,750.00 from the H. P. Cummings Construction Company.

"The work included the removal of a large amount of loose concrete on the outside of the standpipe, the addition of reinforcement for the first 75 ft. and covering of same with concrete by the use of a concrete gun. This work related entirely to the structural strength of the standpipe, and not to its porosity, and to insure a reasonably tight tank, the Western Waterproofing Company put additional moppings of Ironite on the interior of the tank. The Western Waterproofing Company consider part of this as extra work and have made a request for payment. The Board has this matter under advisement. All work has been done to the satisfaction of Messrs. J. R. Worcester & Company and our concrete tank appears to be reasonably devoid of seepage.

"When the work of removing the loose concrete was undertaken, it was found that the exterior had deteriorated more than was anticipated and it was considered wise by the Board and J. R. Worcester Company to remove much more of the exterior concrete than had been contemplated in the specifications. This has resulted in extra work to the extent of $926.91 over the appropriation, and your Board will request at an early Town Meeting in 1922, that an appropriation be made to cover this amount so that the Contractor can be paid.

"The Board feels that events have justified their use of the Ironite method of waterproofing rather than the membrane method, both on account of the cheapness and ease of application, and we further believe that the Town will have a reasonably tight standpipe for many years to come."

1932 "The leakage from the standpipe varies with the temperature, and amounts to from 3000 to 5000 g.p.d. Permanent repairs cannot be made, and I would recommend that no money be expended for this purpose but that a new standpipe of larger capacity be constructed as soon as possible."

A steel standpipe of double the capacity of the concrete one was built in 1934, and since its completion the concrete standpipe has stood empty. Its present appearance is good, and it can doubtless be used whenever it is necessary to paint the steel standpipe.

A summary of annual expenditures for this standpipe follows.

1912—Cost	$19,923.02
1915—Repairs	255.00
1919—Engineer's Report	255.00
1920—Repairs	17,436.44
1921—Repairs	16,349.03
1922—Repairs	1,095.28
Engineers	37.85
1924—Engineer's Report	197.08
Total cost of repairs including Engineering	$35,625.68

Winchester, Mass. This structure is shown in Figure 7. It is 29 ft. in diameter and 43.5 ft. high and was built in 1912. The

FIG. 7—WINCHESTER, MASS., STANDPIPE, 1937.

original appearance of this structure was good.[9] It is not believed that the present structure is in a dangerous condition, but it is unsightly.

Topsham, Me.[10] (Brunswick and Topsham Water District). The diameter of this standpipe is 97 ft. and its height 47.75 ft. In its original condition, the joints between successive day's work were quite marked, and there was considerable efflorescence on the outer surface. With regard to present conditions, Charles L. Bowker, Supt., reports as follows:

"We have always had considerable seepage. So far we have done nothing about it but on the north side we may have to do some work where the exterior is spalled. It does not appear to be serious yet. If you remember, the walls of the tank are quite thick and heavily reinforced, which is a help."

Falmouth, Mass. This structure has a diameter of 40 ft. and a height of 40 ft. It was built in 1913. George A. Griffin, Civil

[9]This Journal, **29**, 190 (1915), Plates 12 and 13.
[10]This Journal, **29**, 196 (1915), Plate 17.

Engineer, reports that it has stood up very well and has never required repairs. It is unsightly because of efflorescence extending down from the joints, but there is no leakage apparent.

Duxbury, Mass. The standpipe is shown in Figure 8. It has

FIG. 8—DUXBURY, MASS., STANDPIPE, 1937.

a diameter of 40 ft. and a height of 35 ft., and was built in 1914. Its general condition is good. Probably no repairs have been necessary. Examination in November, 1937, showed some discoloration by efflorescence, also seepage and wet spots noticeable at several places, all at joints between pourings, and most marked at about 20 ft. above ground and on north side.

ECONOMICS OF PUMPING EQUIPMENT

BY D. D. EAMES*

[*Read January 20, 1938.*]

The subject assigned to me, "Economics Involved in the Selection of Pumping Equipment for Water Works," deals with relative installation costs and operating costs of different types of pumping plants.

Preliminary Considerations

It seems to me that the method of supplying driving power for the pumps is the primary phase of this subject. If we assume that in present-day water works practice centrifugal pumps are the accepted standard type, we recognize three practicable ways of driving them, namely (1) by electric motor using central-station current, (2) by Diesel engine, and (3) by steam engine or turbine. The selection of new pumping equipment for any station almost always involves a study of these three types of motive power, and upon the correctness of the answer reached rests the gain or loss of many dollars over the useful life of the equipment.

General Nature of Discussion. Before developing this phase of the subject in detail, some general considerations will be taken up. It ought to be emphasized at the outset that every installation represents a problem by itself which must be analyzed in accordance with its own particular set of conditions. To follow some general discussion or the practice of some other plant is more than likely to lead to an uneconomical installation for the case in hand. It might also be mentioned that some strictly engineering considerations are so closely related to the economics as to be practically indistinguishable.

Varying Characteristics of Pumping Station Load. The local conditions which determine the best size of pump and the best method of driving it are many and varied. Some of the variations will be as follows:

a. The proposed pumping unit may be: (1) for an entirely new station; (2) an addition to existing capacity; or (3) a replacement of old equipment to reduce cost of pumping.

b. The character of service may be: (1) single pumpage; (2) double pumpage; or some special service such as (3) pumping of well water from the ground.

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*Consulting Engineer, Boston, Mass.

The character of the load usually depends on the reservoir storage capacity.

(1) With large capacity, uniform pumping over 24 hours may be possible, the storage making up for all inequalities of use. In this case the peak load on the station will be that due to seasonal variation, and may be 25% more than the yearly average.

(2) With small reservoir capacity, only sufficient to eliminate short peaks, a moderately varying pumping rate will be required, and the summer peak will be 50% to 60% above the yearly average.

(3) With no reservoir, widely fluctuating pumping rates will be unavoidable. In this case also the dynamic pumping head may undergo only small variations, as where the station is close to center of distribution, and the mains are ample, or it may fluctuate very greatly if there is, for instance, a long transmission line between the station and the center of distribution. In the former instance the summer peak may be two to three times the average, and in the latter it may be four times the average.

Influence of Capacity Requirements. The major condition affecting an installation is, of course, that of size. Here an infinite variety of situations arises, and an arbitrary classification becomes necessary in order to make an analysis.

Definition of Power Unit for Pumps. The power unit of a pumping plant is horse power of load rather than quantity of water pumped. Thus a large quantity against a low head is equivalent in power to a small quantity against a high head, and two such stations are more comparable as to equipment and cost of operation than two stations having similar quantities but widely varying horse power.

Costs for Three Types of Stations

With these preliminary statements, I now propose to compute the comparative first cost and yearly operating cost of four different sizes of stations operated by the three power sources named before; that is, steam power, Diesel power, and central-station electric power. We may then be able to see if any one particular type of plant is best suited to all conditions, or if there are any general conclusions that may be reached. This method of analysis,[1] is not original with the writer, but he is endeavoring to make all assumptions that correspond with conditions prevailing in the New England states at the present time, and is thus endeavoring to arrive at conclusions correspondingly up to date and suited to this territory.

[1]A. P. Pigman, Journal A. W. W. A. March 1930.

The four stations to be analyzed are to lie within the following load groups:

1. Average load of less than 250 hp.
2. Average load of 250 to 500 hp.
3. Average load of 500 to 1,000 hp.
4. Average load of more than 1,000 hp.

Two Assumed Conditions Governing the Estimates. We will assume first that each problem calls for the building of a new plant to pump single pumpage at a relatively uniform rate. The cost estimates for these plants include buildings and complete equipment, with all necessary spare equipment, so that failure of any one piece will not curtail the plant's capacity. This has been carefully carried out except that for stations using central-station electric power no equipment has been included to offset failure of outside current. The customary way to provide such reserve is by gasoline motor-driven units, but I have not included such units, thus giving the electric plant a corresponding advantage in first cost and in the fixed charge portion of operating cost.

For the second part of the problem, I have assumed that new equipment is being installed in existing stations to supplement older equipment. It is assumed also that the old equipment is suitable for reserve or emergency use, that no new building work is required, and that the new equipment will be operated by the same men as for the old plant. This second set of conditions applies more generally to the actual circumstances surrounding pumping-station work than the first set. It is nowadays comparatively rarely that a completely new plant is built.

Further to simplify the task, we will assume for all cases that the total pumping head is 250 ft.; that the cost of coal is $6.00 per short ton; that the cost of Diesel engine fuel oil is 7 c. per gal.; that the cost of lubricating oil is 50 c. per gal.; and that the cost of electric energy averages 1¼ c. per kw. hr., including all demand and energy charges.

In the four sets of studies it is assumed that centrifugal pumps will be used in every case with speed-changing gears for steam and Diesel drives. The only major item omitted from the cost estimates has been the water suction and discharge piping, on the assumption that these costs would be essentially the same for the several sizes of plant and that the comparative cost would be unaffected.

The four ranges of horsepowers into which the size groups are classified have been selected with some reference to the characteristics of steam plants. When a steam plant is large enough to require a tur-

bine of 1,000 hp. or larger, highly efficient turbines are available at low cost per horse power, and the boiler loads are sufficient to warrant stoker or pulverized coal units operating at considerable overload, thus keeping first cost down. The extra cost for high steam pressures and temperatures is also justified in this size of plant. As the size of plant diminishes, more conservative design is necessary, with higher unit first costs and lower operating efficiency. Due, however, to favorable development of boilers and turbines during the last few years, this last consideration is not nearly as weighty as formerly. In Diesel or electric motor plants there is not such a marked variation of efficiency with size of plant. In fact, it is a characteristic of Diesel engines that small units have practically the same fuel rate as the large units.

Table 1 gives the basic engineering data applying to the four

TABLE 1—ENGINEERING DATA AND EQUIPMENT SCHEDULE

GROUP	I	II	III	IV
Normal capacity—m. g. d.	3	6	12	24
Summer peak 25% greater				
Dynamic head—ft.	250	250	250	250
Normal power input to pumps at 84% eff.—hp.	157	312	625	1250
Maximum power input to pumps—hp.	196	390	780	1560
For Steam Plants				
Number of units (1 spare)	2	2	2	2
Maximum capacity of each turbine—hp.	196	390	780	1560
Steam Pressure at throttle—lb.	175	175	225	275
Steam superheat at throttle—of.	100	100	100	100
Steam superheat at throttle—°F				
hp.-ph.	13.1	11.9	10.4	9.6
Maximum boiler output allowing 15% for auxilliary	3,100	5,600	9,800	18,100
Number of boilers and rated horse power each	2—100	2—135	2—200	2—350
For Diesel Plants				
Number of units (1 spare)	2	3	3	3
Normal capacity of each engine—hp.	196	196	390	780
Rotative speed of engine—r. p. m.	600	600	450	300
For Electric Plants				
Number of units (1 spare)	2	3	3	4
Rated capacity of each motor—hp.	200	200	400	500
All motors—2200 volts—1200 r.p.m. Slip Ring				

sizes of plant, together with the number and sizes of each pumping unit. Table 2 gives the first cost and operating cost for each of the three types of plant in the four sizes and on the basis of completely new plants. In the upper portion of the accompanying diagram these

TABLE 2—INSTALLATION AND OPERATING COSTS FOR COMPLETELY NEW PLANTS

	STEAM	DIESEL	ELECTRIC
Group I Investment	$65,600	$46,200	$17,800
Fixed charges 10%	6,600	4,600	1,800
Labor	10,500	6,800	3,000
Fuel or energy	8,100	5,800	14,000
Repairs and maintenance	640	550	300
Total annual cost	25,800	17 800	19,100
Group II Investment	106,800	65,800	26,200
Fixed charges 10%	10,700	6 600	2,600
Labor	14,400	9.000	6.400
Fuel or energy	14,600	11.400	26.900
Repairs and maintenance	1,200	1.100	400
Total annual cost	40,900	28,100	36,300
Group III Investment	156,800	109,600	39.000
Fixed charges 10%	15,700	11,000	3,900
Labor	17,000	11,400	8,500
Fuel or energy	25,200	23.000	54.000
Repairs and maintenance	2,200	2,200	600
Total annual cost	60,100	47,600	67,000
Group IV Investment	236.000	193,800	61,000
Fixed charges 10%	23,600	19,400	6,100
Labor	19.800	14 000	9.000
Fuel or energy	45.000	45.800	107.600
Repairs and maintenance	4,360	4.400	1 300
Total annual cost	92,800	84,000	124 000

twelve operating cost figures have been plotted to show graphically the relation of the operating costs of the three types of plant. The curves indicate that for the larger-size plants steam power and Diesel power are fairly close together, while central-station power is very much higher. For the smallest size plant, central station current appears nearly as economical as Diesel power, and in smaller plants to equal or possibly surpass it. The standing of electric units must depend, for every particular case, on the rate which may be obtained. Consideration must be given to the effect of service during restricted hours, both as to its additional cost and its effect on the continuity of supply.

Table 3 is similar to Table 2, except that it applies to equipment installed in existing plants, as previously explained. In computing first cost, building costs are omitted, as well as spare capacity units. In computing operating cost, the fuel or energy and the repairs and maintenance charges remain unchanged, but the fixed charges are

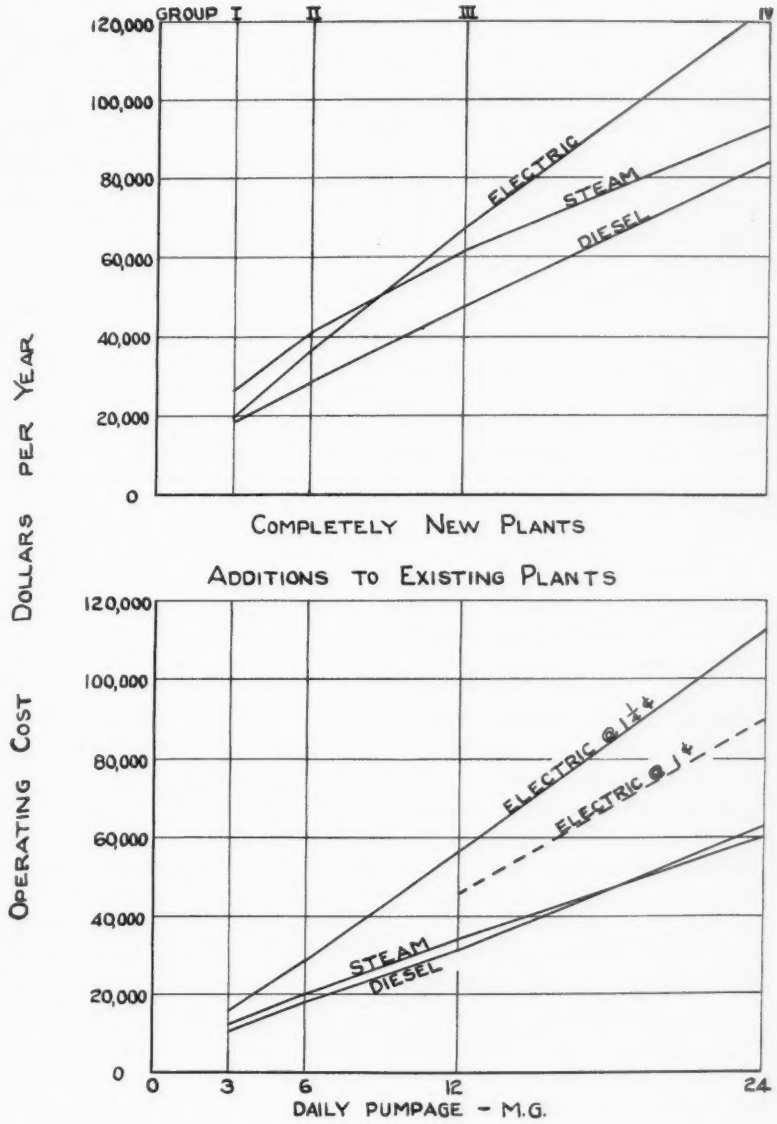
GROUP I
II
III
IV
120,000
100,000
80,000
60,000
40,000
20,000
0
ELECTRIC
STEAM
DIESEL
COMPLETELY NEW PLANTS
ADDITIONS TO EXISTING PLANTS
OPERATING COST DOLLARS PER YEAR
ELECTRIC @ 1½¢
ELECTRIC @ 1¢
STEAM
DIESEL
0
3
6
12
24
DAILY PUMPAGE - M.G.

TABLE 3—INSTALLATION AND OPERATING COSTS FOR ADDITIONS TO EXISTING PLANTS

	STEAM	DIESEL	ELECTRIC
Group I Investment	$27,700	$25,500	$6,550
Fixed charges 10%	2,800	2,600	660
Labor	—	—	—
Fuel or energy	8,100	5,800	14,000
Repairs and maintenance	640	550	300
Total annual cost	11,500	9,000	15,000
Group II Investment	46,600	36,600	13,300
Fixed charges 10%	4,700	3,700	1,330
Labor	—	—	—
Fuel or energy	14,600	11,400	26,900
Repairs and maintenance	1,200	1,100	400
Total annual cost	20,500	16,200	28,600
Group III Investment	68,900	63,800	19,900
Fixed charges 10%	6,900	6,400	1,990
Labor	—	—	—
Fuel or energy	25,200	23,000	54,000
Repairs and maintenance	2,200	2,200	600
Total annual cost	34,300	31,600	56,600
Group IV Investment	102,000	120,700	34,500
Fixed charges 10%	10,200	12,100	3,450
Labor	—	—	—
Fuel or energy	45,000	45,800	107,600
Repairs and maintenance	4,360	4,400	1,300
Total annual cost	59,600	62,300	112,400

reduced, and the labor is omitted entirely. These twelve operating-cost figures have been transferred and plotted in the lower portion of the accompanying diagram. The curves for operating cost on the basis of additional equipment in existing plants show that, for this condition, the steam plants improve their standing with respect to Diesels, and central-station power becomes much poorer with reference to both steam and Diesels. The explanation for this is that, in the case of steam and Diesel power, fixed charges and labor are much larger items of operating cost than for electric units. When these items are substantially reduced, or omitted altogether, the standing is relatively improved. Lest it be said that 1¼ c. per kw. hr. is too high a rate for plants of as large as 12- and 24-m.g.d., I have plotted the cost for 1 c. per kw. hr. for these two sizes. The result is still well above the steam and Diesel costs.

Conclusions

This study seems to indicate that for entirely new plants of the larger size, Diesel-driven pumps are the most economical, but with steam-driven units sufficiently close to warrant most thorough consideration. Electric units for the smaller capacities excel the steam units and approach the Diesel units, with the indication that for still smaller sizes they would surpass both the Diesel and steam units.

For installations of new equipment in existing stations, the Diesel units retain their lead, but with steam units pressing them more closely than before and surpassing them for the largest size. Electric units, on the other hand, have retrograded from their previous standing in all four sizes.

These conclusions may be regarded as correct only for the conditions assumed. A change in any one or more of these conditions may change the entire relationship. The lesson of the paper, therefore, seems to be this: that for any particular installation none of the three types of drive can be arbitrarily ruled out, and that the most economical equipment can only be determined by an analysis based on the exact conditions that which will prevail at the location under consideration.

Appendix

The following tables give the detailed figures for investment cost and operating cost from which the figures in the paper were derived.

Investment Cost for Complete New Stations.

Table 4 Steam stations.
Table 5 Diesel stations.
Table 6 Electric stations.

Operating Cost for Complete New Stations.

Table 7 Steam stations.
Table 8 Diesel stations.
Table 9 Electric stations.

Investment Cost for Equipment *Additional* to existing equipment.

Table 10 Steam stations.
Table 11 Diesel stations.
Table 12 Electric stations.

Operating Cost for Equipment *Additional* to existing equipment.

See Table 4.

TABLE 4—COST OF STEAM STATIONS FOR COMPLETE NEW PLANTS

ITEM	GROUP I	II	III	IV
Boiler, superheater and stoker, soot blowers, boiler brickwork	$11,000	$14,500	$18,500	$28,000
Smoke flue	500	700	900	1,100
Boiler access galleries	200	300	600	800
Two boiler feed pumps	500	800	1,200	1,600
De-aerating feed water heater	800	1,000	1,600	2,000
Steam and water piping	4,000	6,500	9,000	12,000
Piping and smoke flue insulation	900	1,500	2,000	2,500
Combustion control	600	600	800	800
Instruments and meters	200	400	2,000	2,000
Turbine, gear and pumps (2), condenser and auxiliaries, instruments and panel	25,000	48,000	73,000	116,000
Building and stack foundation	9,000	11,000	16,000	25,500
Stack	2,400	3,100	3,800	4,500
Boiler and pump foundations	700	1000	1,300	1,600
Feed water heater gallery	200	400	600	800
Coal storage and handling	1,000	3,000	5,000	6,000
	57,000	92,800	136,300	205,200
Contingencies and engineering 15%	8,600	14,000	20,500	30,700
Total	$65,600	$106,800	$156,800	$235,900

TABLE 5—COST OF DIESEL STATIONS FOR COMPLETE NEW PLANTS

ITEM	GROUP I	II	III	IV
Engines and equipment, delivered and erected and including engine standard equipment and spares, couplings, pyrometers, tachometers automatic stops, silencers, air filters, circulating pump, heat exchangers, expansion tanks, centrifuge, fuel transfer pump, engine piping, foundation and bolts	$23,000	$33,000	$60,000	$117,000
Pumps and gears	8,000	11,400	17,300	27,500
Fuel storage tanks and other piping	1,200	1,800	3,500	7,000
Building	8,000	11,000	14,500	17,000
	$40,200	$57,200	$95,300	$168,500
Contingencies and engineering	6,000	8,600	14,300	25,300
Total	$46,200	$65,800	$109,600	$193,800

TABLE 6—COST OF ELECTRIC STATIONS FOR COMPLETE NEW PLANTS

ITEM	GROUP I	II	III	IV
Motors with primary switcher and controllers	$5,800	$8,700	$13,100	$20,400
Pumps with couplings	4,400	6,600	10,000	15,500
Wiring	600	1,000	1,500	2,000
Building and foundations	4,700	6,400	9,200	14,800
	$15,500	$22,700	$33,800	$52,700
Contingencies and engineering 15%	2,300	3,500	5,000	7,900
Total cost	$17,800	$26,200	$38,800	$60,600

TABLE 7—STEAM PLANTS—OPERATING COSTS

	ITEM	GROUP I	II	III	IV
	Fuel Cost				
1	Av. load on turbine—hp.	157	312	625	1250
2	Turbine capacity—boiler hp.	196	390	780	1560
3	Steam pressure, throttle, gauge—lb. per sq. in.	175	175	225	275
4	Superheat, throttle—°F	100	100	100	100
5	Water rate—full load—lb. per hr.	13.1	11.9	10.4	9.6
6	Water rate average for actual load—lb. per hr.	13.8	12.5	10.9	10.1
7	Steam to turbine—lb per hr.	2170	3900	6810	12600
8	Allowance for auxilliaries and losses—lb. per hr.	430	780	1360	2520
9	Total steam per hour—lb.	2600	4680	8170	15120
10	Heat in steam at boiler—B.t.u. per lb.	1269	1269	1276	1279
11	Heat of liquid, temperature of exhaust—°F	48	48	48	48
12	Heat added in boiler, B.t.u. per lb.	1221	1221	1228	1231
13	Heat value of coal as fired—B.t.u. per lb.	14200	14200	14200	14200
14	Overall boiler efficiency—%	72	72	74	76
15	Evaporation-lb. water per lb. coal	8.4	8.4	8.55	8.9
16	Coal burned per hour—lb.	310	560	960	1725
17	Coal burned per day—tons	3.7	6.7	11.5	20.6
18	Coal burned per year—tons	1360	2440	4200	7500
19	Cost of coal per ton	$6.00	$6.00	$6.00	$6.00
20	Annual cost of coal	$8,100	$14,600	$25,200	$45,000
	Labor				
	Chief engineer	—	1	1	1
	Assistant chief	1	1	1	1
	Watch engineers	2	2	2	2
	Watch firemen	3	3	3	3
	Relief engineer	—	1	1	1
	Relief fireman	1	1	1	1
	Coal passer and ashes	—	—	1	2
21	Number of operators	7	9	10	11
22	Average yearly wage	$1500	$1,600	$1,700	$1,800
23	Annual labor cost	10,500	14,400	17,000	19,800
	Maintenance & Repairs				
24	Boiler room	$300	$520	$ 880	$1,620
25	Pump room	340	680	1,370	2,740
26	Total	640	1,200	2,250	4,360

TABLE 8—DIESEL PLANTS—OPERATING COSTS

		Group I	II	III	IV
	Cost of oil per gal.—c	7			
	Weight of oil per gal.—lb.	7.5			
	Fuel consumption at average load—lb. per hp.-hr.	0.4			
	Cost of oil per hp. hr.	0.00374			
	Add 12% for lubricating oil	0.00045			
		0.00419			
1	Average load on engines	157	312	625	1,250
2	Horse power hours per year	1,374,000	2,730,000	5,460,000	10,920,000
3	Fuel & lubricating oil per hp.-hr.—c	.419	.419	.419	.419

TABLE 8 (Continued)

4	Cost fuel & lubrication oil per year	$5,800	$11,400	$23,000	$45,800
5	Number of operators	4	5	6	7
6	Average yearly wage	$1,700	$1,800	$1,900	$2,000
7	Annual labor cost	6,800	9,000	11,400	14,000
8	Maintenance @ $0.004 per hp.-hr.	550	1,100	2,200	4,400

TABLE 9—ELECTRIC MOTOR—OPERATING COSTS

		I	II	III	IV
1	Average horsepower pump input	157	312	625	1,250
2	Maximum horsepower pump input	196	392	780	1,560
3	Average horsepower corrected for omission of gear	154	306	613	1,225
4	Average motor efficiency—%	.90	.93	.93	.93
5	Average motor horsepower input	171	329	658	1,316
6	Average motor kilowatt input	128	245	492	983
7	Kilowatt hours per year	1,120,000	2,150,000	4,320,000	8,610,000
8	Cost per kw.-hr.—c.	1¼	1¼	1¼	1¼
9	Cost per year for energy	$14,000	$26,900	$54,000	$107,600

TABLE 10—COST OF STEAM EQUIPMENT FOR ADDITIONS TO EXISTING PLANTS

	GROUP			
	I	II	III	IV
Boiler room equipment	$10,000	$14,000	$19,000	$26,000
Pump room equipment	13,000	24,000	37,000	58,000
Boiler and pump foundations	400	600	800	1,000
Feed-water-heater gallery	200	400	600	800
Coal handling	500	1,500	2,500	3,000
	$24,100	$40,500	$59,900	$88,800
Contingencies and engineering 15%	3,600	6,100	9,000	13,200
Total	$27,700	$46,600	$68,900	$102,000

TABLE 11—COST OF DIESEL EQUIPMENT FOR ADDITIONS TO EXISTING PLANTS

	GROUP			
	I	II	III	IV
Engines, pumps and equipment	$21,000	$30,000	$52,000	$98,000
Storage tanks and piping	1,200	1,800	3,500	7,000
	$22,200	$31,800	$55,500	$105,000
Contingencies and engineering 15%	3,300	4,800	8,300	15,700
Total	$25,500	$36,600	$63,800	$120,700

TABLE 12—COST OF MOTOR EQUIPMENT FOR ADDITIONS TO EXISTING PLANTS

	GROUP			
	I	II	III	IV
Motors	$3,000	$6,000	$9,000	$15,500
Pumps	2,200	4,500	6,800	12,000
Wiring	300	700	1,000	1,500
	$5,700	$11,600	$17,300	$30,000
Contingencies and engineering 15%	850	1,700	2,600	4,500
Total	$6,550	$13,300	$19,900	$34,500

DIESEL PUMPING PLANT AT MILFORD, MASSACHUSETTS

BY M. H. GOFF*

[*Read March 17, 1938*]

In 1881, the first pumping engine was installed at the Milford Water Company. This was a Holly quadruplex pump of 1,500,000-g.p.d. The Holly Company installed the plant complete with two horizontal tubular boilers. In 1888, a Knowles horizontal duplex compound was installed with 750,000-g.p.d. capacity. In 1890, a Worthington compound duplex pump was purchased. All three of these steam engines have been removed at different intervals.

In 1900, we installed a Holly Gaskill type high-duty pumping engine of 3-m.g.d. capacity. This served us for thirty-six years and gave good service during that time. In 1925, we purchased a turbine-driven centrifugal pump. This was removed in 1937, along with the Holly Gaskill pump. The turbine was used only as an auxiliary pump. This pump was considered adequate in case of emergency, but not for continual operation.

In 1934, extensive repairs on the Holly Gaskill pump were needed. In this connection the advisibility of installing an internal combustion engine was considered. Electric pumping was studied, but the rates prevailing in our locality were so high as to make pumping costs excessive. Diesel engines had been given much consideration by the Company for two or three years previous to the first installation.

In October, 1933, contact was made with the Worthington Pump and Machinery Corporation of Boston, who sent to Milford their engineer to investigate conditions as to dynamic head and vacuum of our present distribution system. This being the first Diesel engine to be installed by the Water Company, it was thought necessary that this pump should run to capacity in order to give high efficiency. On November 24, 1933, we contracted to buy one Worthington 100 hp. direct-injection Diesel engine. After a complete study of our system, it was necessary to install a two-stage series-connected centrifugal pump, to be directly driven by this Diesel engine. The engine operates at a speed of 514 r.p.m., driving direct to a speed-increasing gear unit of herring-bone gears with a resulting pump speed of 1750 r.p.m. The total dynamic head is 286 ft., and the suction lift averages 11 in. of vacuum. The capacity of the 100-hp. unit is 1040 g.p.m.

*Assistant Superintendent, Milford Water Co., Milford, Mass.

The pumping facilities at this time included the turbine-driven 3-m.g.d. pump and also the 3-m.g.d. Holly high-duty pumping engine of the Gaskill type. Under steam operations, it was interesting to note on the Venturi chart, considerable variation as to the amount of steam in the boilers. With the Diesel engine in operation, the line on the chart did not vary, a straight line showing at all times when pumping. This indicated very even pressure in our distribution system which was of benefit to the mains. The results of this little engine were so satisfactory for one year that it was decided to purchase another Diesel engine of larger horse power to aid in pumping.

In September, 1935, the Milford Water Company purchased its second Diesel engine of 125-h.p., 5-cylinder, 4-cycle type driving a two-stage series-connected centrifugal pump. This installation gave us 1300 g.p.m., the engine speed remaining the same. After an operation period of six months, this unit indicated that a 5-cylinder proposition was much more satisfactory for our pumping needs. This unit had the same usual gear box, turning the pump at 1750 r.p.m. with engine revolutions the same as those of our first small Diesel engine. Our exhaust line was arranged so that it passed through a specially constructed tank. When this unit is running, we have hot water at a temperature of about 140° F. At this time the boiler fires were extinguished and have never been lighted since. This left the pumping station with two steam pumps that were not to be used again.

After three years of Diesel pumping, it was decided that we should use Diesel power exclusively and discard all of the steam pumping equipment. With only two Diesel pumping engines in the station we felt that we had only one stand-by engine in case of break-down. This condition would leave us without adequate fire protection. For this reason a third Diesel engine was installed. This is a 6-cylinder Worthington 180-h.p., 4-cycle engine direct-driven to the usual increasing gear unit. This third engine operates at 600 r.p.m., but the two-stage pumps are driven at the same speed, namely 1750 r.p.m. The valves, pistons, piston rings and heads of these three engines are interchangeable. The efficiency of this arrangement cannot be over-estimated, since we are thus at all times protected against a prolonged shut-down.

In 1937, when the 6-cylinder unit was installed we purchased a Gould centrifuge for cleaning the lubricating oil in the crank case of the Diesels. The centrifuge is connected to all three engines and can be operated independently of each. We have been told that lubricating oil can be used without changing it for twelve to fourteen

months, providing we "wash" the oil once a day. The capacity of this centrifuge is about 40 gal. per hour. The lubricating oil in the crank case of the small engine, holding 11 gal., can be centrifuged nearly four times per hour. This centrifuge has been in operation for about six weeks, and the results are very satisfactory. It removes all carbon, sludge and vapor from the crank case, leaving the inside nearly as clean as when new. The consumption of lubricating oil in our 4- and 5-cylinder engines has averaged one gallon to every 4,000 hp.-hr. In 1937, 31,991 gal. of Diesel fuel oil were consumed in pumping 311,990,000 gal. of water supplying 3,380 services. The average daily pumping was 850,000 gal. The largest day's consumption was 1,400,000 gal.

The first and smallest Diesel engine serves us very well for winter months with a rating of 1,5000,000 g.p.d. The second engine, the 5-cylinder Diesel is adequate for spring and some summer days with a rating of 1,850,000 g.p.d. The latest Diesel engine is capable of serving us at all times with a rating of 2,900,000 g.p.d. This is more than sufficient for heavy summer-day consumption and any fire emergency that may arise. Each Diesel averages a daily run of 14 hr.

Water is taken from the discharge line so that we always have an adequate supply running through the cooling systems of these three Diesel engines. The cooling water is discharged through a sight glass to a natural drain. We have discovered that we can start one of these engines three times on 250 lb. of compressed air without recharging the tank. Air is furnished by a small electrically-driven, single-stage air compressor and a gasoline-driven, two-stage air compressor for stand-by. Located outside the station in the ground is the 10,000-gal. fuel oil tank. This oil is pumped daily into a 150-gal. "day tank", the fuel oil running to each engine under gravity.

When the boilers were discontinued it was necessary to find means of heating the station through the winter months. A small oil burner was purchased which burns the same grade of oil as the Diesel engines.

The installations have been an asset to the Water Company in many ways. Pumping costs have been greatly reduced, both for fuel and labor. We have a clean attractive station, systematized pumping service, quicker pick-up in emergencies and increased efficiency at all times.

PRINCIPLES OF CORROSION AND ITS PREVENTION

BY FRANK N. SPELLER*

[*Based on Remarks of September 22, 1937.*]

This Association has done much to promote the study of corrosion. My assistant, in looking over the records, found twenty-nine papers in this journal that dealt more or less with the corrosion problem. These were presented before this Association during the past ten to twelve years.

The subject of corrosion is increasing in interest all over the world. Last year we had four symposiums in English that dealt, directly or indirectly, with that subject, and it is now one of the major engineering problems of the world. About thirty years ago, many thought that this was only one comparatively simple problem, and that all that was needed was to find the metal that would withstand water conditions and the problem would be solved. We now know that corrosion consists of a multitude of problems, and that while the underlying principles are comparatively simple, there are so many factors involved that it is an exceedingly intricate question. The general conclusion is that there is no one metal, or one coating, or one treatment, that can ever solve all the various corrosion problems.

The principles of corrosion are very simple, but the practical application is another matter. Metals are unstable. They have been won from their ores by the expenditure of energy, and they tend to revert to compounds that are more stable with the release of free energy. If we remember that, we have learned a whole lot about corrosion. The wonder really is that metals last as long as they do. It is a lot easier to explain why they corrode than to explain why in some cases they do not corrode.

Nature sometimes provides restraining influences in the surface reaction between the metal and its environments, and these surface reactions tend to form protective films. We know, for instance, that most metals last a very long time in some environments, as in certain soils and atmospheres, and a relatively short time in others. Furthermore, we have learned a great deal about the protection of metals from the study of these natural protective agencies. The basis of alloy rust-resisting steels is the film theory, and nature has shown us the way.

Last summer I was in Northern Canada and a farmer we met had

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some iron wagon wheel tires, that were in a splendid state of preservation after twenty years out-of-doors. They had a surface film that reminded me of the film that is often seen on copper steel. The metal apparently acquired that film because of the purity of the air in that district, the freedom from destructive dirt, and the rather long periods of low humidity with strong sunlight. All these things tend to build up such a film, and under such conditions neither copper nor chromium or anything else is needed in iron or steel. In our Pittsburgh atmosphere, that iron would have corroded seriously in much less time.

There is corrosion by direct chemical attack or by electro-chemical action. Examples of direct chemical corrosion are the oxidation of iron at high temperature, or the rusting of iron in pure dust-free air. But you are more interested in water and soil corrosion, and these are almost entirely electro-chemical phenomena, so that I am going to talk about that phase of corrosion alone.

Suppose we have a vessel of water containing a salt solution and we put in two pieces of metal, for example zinc and iron. A current will flow through the solution from the zinc to the iron. If we replace the zinc with another piece of iron, we will still get a current one way or the other. If there is a little copper in solution, the copper will be deposited out on the cathodic member. If there is no copper in solution, we get hydrogen on the cathode; that is the first reaction of the corrosion process of the water.

When iron goes into solution, something must leave the solution, and if copper is there it can be seen deposited on the metal. The hydrogen cannot be seen, but we all know that hydrogen acts like a metal, and it is plated out on the cathode.

So these are the fundamental reactions of corrosion in water. However, we must consider the tendency to corrosion, the rate of corrosion, and its distribution. These are entirely different things governed by different factors. The fundamental process is the solution of the iron in water, with the liberation of the hydrogen.

If oxidation occurs at the cathode, hydrogen is removed, more is deposited, and the process goes on, but if there is something in the solution that will form a deposit on the anode, anodic polarization occurs and corrosion is restrained in another way. For instance, the chromates will form a film on the anode or, if lime is present, and there is no equilibrium, it may be precipitated mainly on the cathode and thus interfere with diffusion of oxygen to the metal. The well-known precipitation of lime in water pipe is an example of this reaction. It is much safer to have cathodic protection than anodic protection, because with *incomplete* anodic protection, as with chro-

mates in water, it is sometimes worse than no protection at all. When the relative area of the anode is reduced, the rate of penetration of the metal increases.

I have pictured this as a battery reaction, but we have the same thing, let us say, on the surface of a piece of pipe. Certain areas will be anodic, and others cathodic. Exactly the same reaction occurs on the surface of metal in wet soil as in a wet battery. In other words, corrosion in soil and water are essentially electrolytic processes.

The driving force of the original reaction is the solution pressure of the metal. The factors that keep the process going are secondary, but very important. These include film formation either at the anode or the cathode, and such processes as depolarization by oxygen, rate of motion of water, and temperature. Secondary reactions may affect the anodic or cathodic reactions, but controlling cathodic reaction is safer than depending on anodic polarization. It is not necessary to do both. It is like controlling flow of a fluid in a pipe line with two valves; both are not needed to restrict the flow.

In localization of corrosion, another set of factors is involved. In this case, conductivity affects the action of dissimilar materials. Differences in concentration of solutions on the surface of the metal cause a different potential, particularly differences in the concentration of the oxygen. The portion of the metal which is exposed to the higher oxygen concentration is always cathodic. In our illustration, if there were no difference of potential and we lifted one of the pieces of iron out of solution for an instant and then put it back again, we would immediately get a much stronger current.

Velocity is another important factor. These factors do not always work the same way. Oxygen, for instance, increases the rate of corrosion by depolarizing the metal in solution, but in the atmosphere, under certain conditions—for instance where the humidity is rather low—oxygen builds up a protective film and retards corrosion. The effect of velocity is essentially to bring the oxygen more intimately in contact with the cathodic surface. If oxidation becomes very active (by high velocity, for example), a gelatinous ferric hydroxide film forms which is highly protective. If we can get the corrosion products to deposit where they are formed, they are very likely to stifle the reaction and protect the metal; but, if they wander away, the ferric hydroxide will form perhaps a quarter of an inch away from the place where the solution of iron occurs, and eventually tubercles of rust form on the surface.

High velocity tends to keep the corrosive products near their source. If the velocity is increased still further, to a very high rate, the metal may be cleaned by erosion. We can't say that any one

factor has a certain effect because each factor is often interdependent with others.

In regard to protection, I will just mention the three principles on which protection depends. In the first place, the resistance of the metal may be improved. This is based on alloying the metal with certain elements that tend to build up protective films in certain environments. Of course, fundamentally it is practically impossible to make a metal that will act equally well under all environments, because the environment has just as much to do with the formation of the film as the metal itself.

It does not matter very much whether you put the essential film-building elements in the environment (in water, for instance) or in the metal. Chromium or phosphorus, in certain combinations in solution, form protective films on plain iron. The film is likely to be more permanent when formed mainly by the metal.

The next system of protection involves the placing of artificial coatings on the metal. We could spend a whole morning discussing the improvements in paint, particularly for under-water protection. It is surprising what progress has been made in the last few years. Some of the newer synthetic paints of the chlorinated-rubber, phenol-formaldehyde type, and others, are very useful in the protection of metals in air and water, and perhaps for use under ground also. Then there are the portland cement concrete, and bituminous coatings.

Colloids and larger particles of soil have a very interesting effect in retarding the action of corrosion. For instance, in drilling oil wells, the rotary drilling process depends on the fluid mud formed for lubrication. We have had occasion to study this, as it is such an important factor in oil-well drilling. Static corrosion in this mud, which is of the consistency of thick cream, is from one-tenth to one-twentieth as much as in the water with which the mud is made. Water-works engineers know that, before filtration came in, the sediment in the water was highly protective so that the rate of corrosion of pipe was less than after filtration.

The third protective system is the treatment of the environment. There are some things that have not yet been fully studied even in the National Bureau of Standards' test; for example, the effect of trench fill. Promiscuous fill leaves a non-uniform soil structure, so that the oxygen can penetrate quicker through one avenue than another, and strong electrolytic conditions are the result. I believe that a lot of work could be done in studying the economics of trench fill, to get a more uniform soil condition around the pipe. In our mill yard, for example, where pipe is laid in cinders, we have made a practice of putting six inches of plastic clay around the pipe which

was also coated with a thin asphalt dip, and there is no trouble under such conditions with our water pipes.

Cathodic protection of pipe lines is coming very rapidly into use in the oil and gas fields, and it must be a comfort to engineers to know that if a protective coating fails under most conditions you can hitch up a windmill and impress a current on the pipe, and prevent the solution of the metal.

CATHODIC PROTECTION

Combating Pipe Corrosion with Electricity

BY A. V. SMITH*

[*Read January 20, 1938*]

Corrosion of the surface of pipes buried in the soil produces current flow from the surface of the pipe to the soil or water surrounding the pipe. By properly controlling this electrical current, corrosion of pipe may be reduced. This is accomplished by a system which has been termed "Cathodic Protection."

Soil Corrosion

To understand Cathodic Protection one must understand soil corrosion. While this phase of the problem has many different, complicated and insidious causes, the general theory is fairly simple. I propose to start at the beginning of the general theory and explain it without the complicated laws of physics and chemistry.

Current Flow. We all know that water runs down hill. Figure 1 shows a dam with water flowing over the dam to a point of lower level. Electric currents which are involved in soil corrosion flow in a similar manner, that is, they flow from the point of highest potential (level) to the point of lower potential (level). In the case of the storage battery shown in Figure 2, the current flows through the light connected to the battery from the binding post of the highest potential, marked (+), to the binding post of the battery marked minus (—). If we place a plus (+) mark over the water above the dam, and a minus (—) mark over the water at the lowest point below the dam, we can say that the water flows from (+) to (—). This is the same way direct current flows, from (+) to (—).

What happens when a pipe starts to corrode? If the pipe is corroding, the pipe is at the point of highest potential or plus (+) and the earth or water surrounding the pipe is at a lowest potential or minus (—). In Figure 3 is represented a piece of pipe metal subject to corrosion. The object at the left (A) shows current as represented by the arrow starting to leave the metal to flow into the soil. As it goes it hitches itself to a very minute particle of iron (called iron ion) and drags it free of the pipe as shown in (B). The instant it is free a small globule of air (called hydroxyl ion) rushes

*Electrolysis Engineer, The United Gas Improvement Company, Philadelphia, Pa.

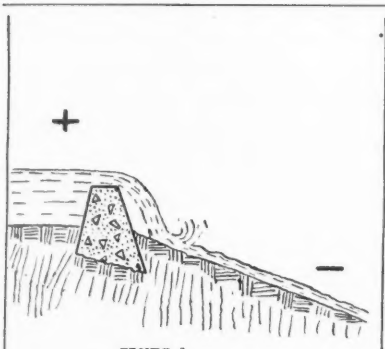

FIGURE 1

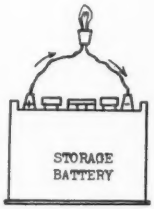

FIGURE 2

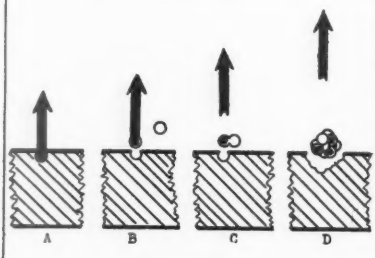

FIGURE 3

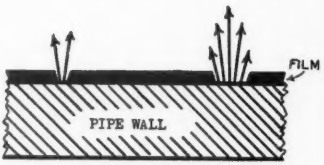

FIGURE 4

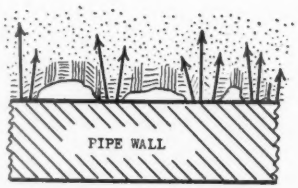

FIGURE 5

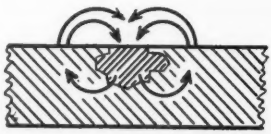

FIGURE 6

over and grabs the small piece of iron from the electric current. The two join together (C) and appear as the rust on the outer surface of the pipe. The chemist calls it iron oxide. This action is continuous as long as current flows and the ions or pieces of iron come from the pipe. If this action continues for a long enough period of time when the oxide scale is brushed or scraped from the pipe a definite pit will be found. The accumulation of oxide and the formation of a pit are shown in (D). There are many different actions when other

particles are waiting outside the pipe to catch the small particle of iron, and many other things occur to the embraced particles, but for this explanation of corrosion they are incidental, for they result in a pit in much the same way as described.

Fundamentally then we shall say that wherever the current flows into the soil it will drag a small piece of iron from the pipe. Why does a pipe pit? It does not corrode uniformly all over, because the current doesn't leave the metal surface at all points. Figures 4 and 5 show why the currents do not leave the pipe over the entire area or surface. Figure 4 represents a pipe that has had a film of protective oil, varnish or paint applied at the mill to prevent rusting in storage or during transit. This film tends to resist the current much as a dam resists or retards the flow of water. Therefore, to get free of the pipe it is easier for the current to flow through imperfections in this film than it is to force its way through the film resistance. The result is that the small pieces of iron are dragged from the points where there are film imperfections and not from the part covered with the mill protective coating. Therefore, pits appear only at random places.

Figure 5 represents the pipe contact with the soil. It will be noted that the particles of soil do not actually touch the pipe at all places. The voids offer resistance to current flow as does the film in Figure 4. Current must travel the path of least resistance, so it shuns the voids and flows out only at certain places, with the same effect as the mill protection, that is, with pitting and not with uniform corrosion. Combine both of the conditions, in Figures 4 and 5, which gives the condition of a buried pipe, and it will be understood that it would be a mystery if the pipe did not pit.

Now that it is known what the current does,, we must consider what causes the current to flow. There are two general causes of current flow: stray or vagrant currents from electric street railways, and currents generated by local conditions, much as a storage battery "generates" current, and called galvanic current. Corrosion by electrolysis is called corrosion due to stray railway currents and soil corrosion is called corrosion by galvanic current.

Galvanic Currents. Since cathodic protection is used to reduce galvanic corrosion, I will discuss galvanic currents. Figure 6 shows galvanic currents generated by impurities in a metal or by dissimilar granular structure of a metal. Local currents flow from the pipe into the soil, from the soil to the impurity, and back through the impurity to the pipe again. Figure 7 shows a cinder resting against the pipe and the currents generated locally. This condition is quite serious and will penetrate the pipe, especially if there are numerous

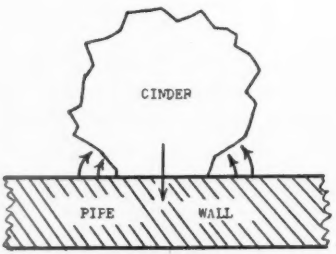

FIGURE 7

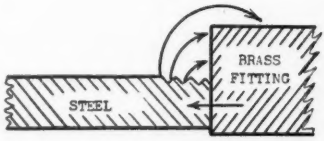

FIGURE 8

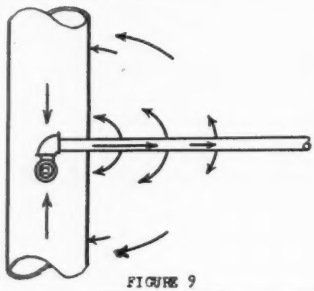

FIGURE 9

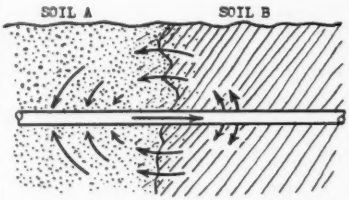

FIGURE 10

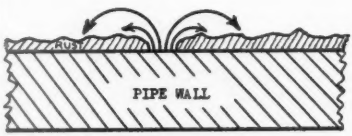

FIGURE 11

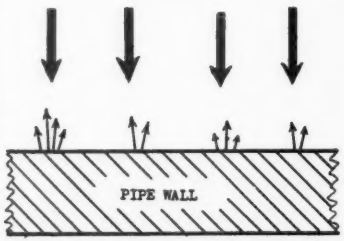

FIGURE 12

cinders in the soil. The remedy, of course, would be to keep the cinders away from the pipe, but this is not always practicable. The next best remedy is to apply a good pipe coating, one that will withstand the soil pressure of the rough cinders. A good coating would be a wrapped coating with asbestos felt wrapper, or encasing the line in lime, concrete or pitch. The wrapped coating appears more practical.

Figure 8 shows the galvanic currents generated by two dissimilar

metals in contact in the soil. Currents flow from the iron pipe through the soil to the brass fitting, returning through the metal contact of the two metals. In this case the iron is corroding. The relative size or the relative area of iron and brass are the determining factor in the amount of iron corroded. If the iron is of large area and the brass small in area, such as a brass shut-off cock in a steel service, the iron corrosion will be found very close to the brass fitting. If there are exposed threads on the iron close to the brass, penetration will probably occur. If the relation of iron and brass are such that a small iron fitting is placed in a brass or copper piping system, destruction of the small iron fitting will probably occur. Figure 9 represents the currents generated by what are, in theory, two dissimilar metals, that is, a new service pipe connected to an old rusty main. The current flows from the new service pipe, causing it to pit, through the soil to the old rusty main. The current then flows along the main to the service which completes the circuit.

Figure 10 shows the galvanic currents generated by a pipe line passing through two dissimilar soils. The currents will flow from the main in one soil, through that soil to the second soil where it returns to the main at that point. It then flows along the pipe line back to the first soil. Figure 11 shows galvanic currents produced by rust forming on parts of the pipe in the soil or by differences in oxygen film.

The review of the general theory of corrosion and how galvanic currents in general may be produced leads to the question of how corrosion may be eliminated. We are to consider Cathodic Protection as a means to this end.

Cathodic Protection

First, what is a cathodic protection? We have seen that current leaving the pipe surface is the result of galvanic action and corrosion. If the current can be prevented from leaving the pipe surface, corrosion will stop.

We know that if a force is acting to move, for example a wagon down hill, the only way we can make the wagon move up hill is to apply a *stronger* force in the *opposite* direction, or up-hill. This is the principle of cathodic protection and is represented in Figure 12. The small arrows at the pipe surface represent the galvanic currents of corrosion. If a stronger current (force) is applied in the opposite direction as shown by the larger arrows, the small currents will be overcome and corrosion will be arrested.

The term "cathodic protection" was actually a technical explanation of what I have told you. Since you know what it is, I will

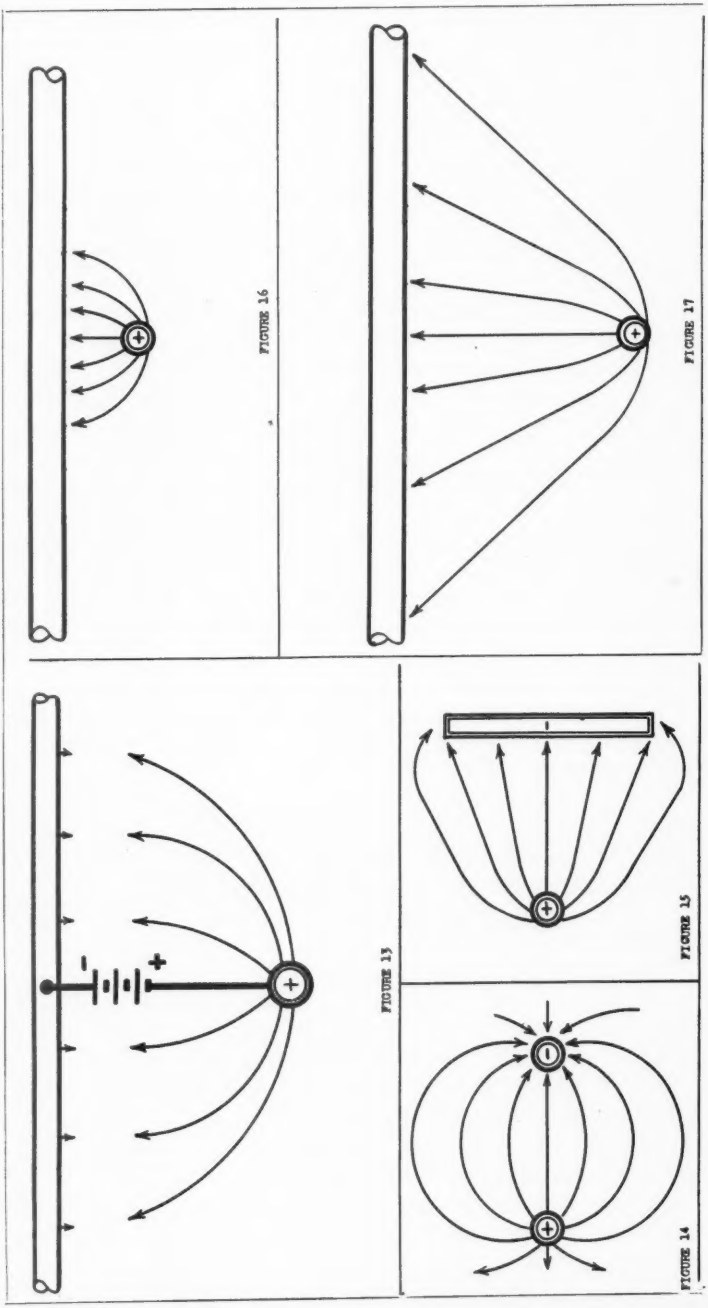

FIGURE 13

FIGURE 14

FIGURE 15

FIGURE 16

FIGURE 17

record here the technical explanation. When a piece of metal corrodes, current leaves the metal to flow through a solution, called an electrolyte, to another piece of metal. The pieces of metal are called electrodes. The metal from which the current flows into the solution is called the "anode" and the piece of the metal to which the current is flowing is called the "cathode". Corrosion occurs where the current is leaving the metal and is therefore anodic corrosion. Corrosion, except in very special cases, does not occur where the current flows *to* a metal, that is, the corrosion does not occur at the *cathode.* Therefore, if we can cause the anode to become a cathode, *i.e.*, if we can cause current to flow to the metal instead of from it, corrosion will be stopped. Therefore, the phrase "cathodic protection"—protection by changing an anode to a cathode—was used to describe such a method.

Pipe Line Protection. How is this theory applied to pipe lines? Figure 13 shows the method of application. The small arrows represent the galvanic currents of corrosion. A battery is connected between the pipe line and a piece of metal buried in the soil at a point some distance from the pipe line. The battery is so connected—[the minus (—) on the pipe line and the plus (+) on the buried metal]—that the battery current will flow to the buried metal, through the soil to the pipe line and thence back to the battery. The battery current is represented by the larger arrows and these overcome the smaller arrows and prevent corrosion.

Now that we know the theoy of corrosion and the theory of applying cathodic protection, what is the best way to make a practical application? First we must consider just how the currents flow in the soil.

If two pieces of metal of the same size are placed in an electrolyte (soil) and there is a potential difference between them, current will flow from the one of highest potential to the one of lowest potential, as shown in Figure 14. The lines are the same as the lines of force of the magnetic field of a horseshoe magnet looking at the ends of the magnet. Figure 15 shows the current path between two pieces of metal of different sizes, the small piece being of highest potential and the larger piece being of lowest potential. Notice how the current lines spread out to cover the large metal and do not converge as shown in Figure 14 with the same size of cathode (—) as the anode (+). So if we bury a small piece of metal close to a pipe line and force current to flow from the buried metal to the pipe line, the current will spread out from the buried metal and cover a definite area of the pipe line as shown in Figure 16. If we bury the piece of metal farther away from the pipe line, the current will

spread out farther and cover more of the pipe length, as shown in Figure 17.

This then is the first principle to consider. We now see that if the buried metal is placed farther away from the pipe line, more of the pipe line will be protected with the same value of current. With the buried metal farther away from the pipe line, it will require only a slightly higher force to make the same current reach the pipe line. But at the same time more pipe area is protected, so that the practical result is that more area can be protected at only slightly higher cost.

It will be found, with one piece of metal buried close to the pipe, that increasing the amount of current will increase the area of protection only within narrow limits. A single piece of metal buried some distance from the pipe line will give better protection at a lower cost than one placed near the pipe line. The cost is further reduced by the use of pipe coating.

Ford, Bacon and Davis, Inc., have patented a method of having the buried metal close to the pipe line and giving the needed protection at lower cost by using more than one electrode spaced at predetermined intervals along the pipe line. This principle is shown in Figure 18.

Protection of Distribution Systems. Cathodic protection has been used successfully on single pipe lines in areas where other pipe lines were not involved. The principle of cathodic protection has also been used for distribution systems subject to electrolysis from stray railway current, and a few engineers have applied cathodic protection to underground structures in a distribution area. The problem of cathodic protection to a pipe line at a location where other underground structures are not connected to the pipe line by good metallic connections requires careful study and cautious procedure. Figure 19 shows what happens where two pipe lines, which are not connected together, are involved. Cathodic protection is applied to pipe line A. Note that a considerable amount of current leaving the buried metal anode flows to pipe line B, travels along this line for a short distance and then discharges to the soil to flow to pipe line A. Pipe line A received cathodic protection at the expense of pipe line B, the latter corroding where the current leaves the pipe.

The way to prevent this is thoroughly to know the facts before applying cathodic protection. Perhaps pipe line B did not require protection in this area because it is some distance from pipe line A and in a different soil. If the buried metal for protecting A is to be placed far enough away from A to obtain good results it will be too near B and cause corrosion of B. To prevent this, pipe line

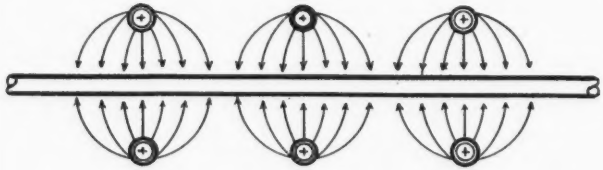

FIGURE 18

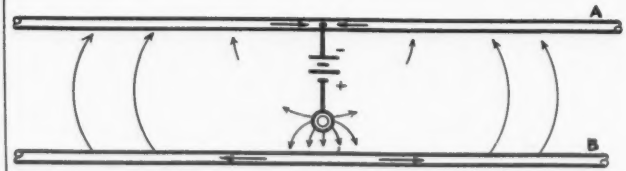

FIGURE 19

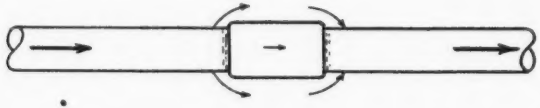

FIGURE 20

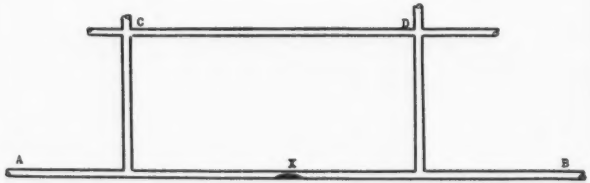

FIGURE 21

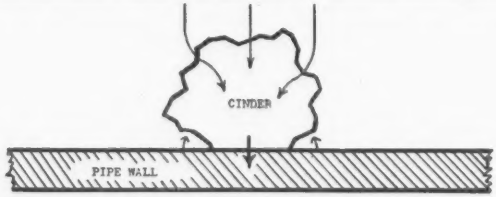

FIGURE 22

B should be included in the cathodic protection, or if the two lines are far enough apart buried metal anodes should be placed close to pipe line A so as not to affect pipe line B. If B is included in the cathodic protection and did not need protection originally, the cost of protecting pipe line A with B included will be appreciably higher.

In a city street where there are numerous structures close together, placing the buried metal anode close to the line to be protected will affect the other structures because of their proximity. It will be necessary in city streets to include all structures in the system of cathodic protection. In city streets where there remain abandoned trolley rails, the trolley rails could be used as the buried metal anode, and all the other underground structures tied metallically together and included in the cathodic protection system.

Applying cathodic protection will increase the amount of current flow on the cathodically protected pipe line. Any abnormal resistance in a pipe line carrying current will produce a hazardous condition. Figure 20 shows a screw joint in a pipe line. If this joint has a slight amount of resistance—the majority of joints of this kind have been found to have more resistance than an equivalent length of pipe without such a joint—some current will shunt around the joint through the soil as shown by the arrows, producing corrosion of the pipe on the side where the current leaves to shunt around the joint.

This may be better understood by the analogy of a water system as represented in Figure 21. If there was a slight stoppage in the main at point X, more water would flow from A to B by way of C-D than if there was no stoppage. The path ACBD represents the soil path in Figure 20 while AB represents the path with the joint in Figure 20 at point X in Figure 21.

Any abnormal resistance such as occurs at screw joints, Dresser couplings, cast-iron mains with Leadite or Hydrotite joints, lead joints in cast-iron pipe, mechanical joints in cast-iron pipe, valves, etc., must be reduced by installing heavy copper bonds around such joints. This would involve considerable expense in the case of cast-iron mains with lead joints at intervals of 12 ft. along the pipe. Mains with cement joints and some mechanical joints offer sufficient resistance to prevent flow of galvanic currents generated by widely separated soils, etc., and confine the corrosion to that due to the soil surrounding each section of main.

In a city where there are liable to be cinders against a pipe, cathodic protection may not stop the corrosion at the cinder. Unless the cathodic currents are very dense and of sufficient magnitude,

the currents will flow as shown in Figure 22. This question does not arise in the case of a steel line with a wrapped coating, for the cinder cannot touch the pipe, and the cathodic currents reach the metal pipe by flowing through pinholes or abrasions if any exist.

Distribution mains of bare cast iron give a fairly long life in most soils where electrolysis is not present. It is doubtful whether or not cathodic protection of such mains is necessary or justified.

High-pressure steel distribution mains do require protection. If these lines are bare and are in city-street networks, cathodic protection will be costly to install, for all structures including cast-iron gas mains must be taken into consideration in the installation. A study of the problem may show that it would be cheaper to isolate the steel main, apply a good asbestos-felt, wrapped coating and break up the flow of galvanic currents by the use of insulating joints. This, of course, applies to locations where electrolysis is not involved. If the problem of stray-current electrolysis is involved, the remedial measures applied to prevent electrolysis will also reduce soil corrosion.

Conclusion. In conclusion, cathodic protection may be applied to isolated transmission lines, or pipe lines, that do not involve other underground structures. In city distribution systems cathodic protection may be applied successfully only in special cases and where proper consideration has been given to all the factors involved.

ENFORCING PAYMENT OF WATER BILLS

BY DANIEL M. SULLIVAN*

[*Read March 17, 1938*]

In the past seven years the expenditures of the Boston Water Department have annually been in excess of departmental receipts. Since 1931, the total deficit has roughly been $747,000. This deficit has been financed (1) by the absorption of a surplus built up over previous years, amounting to $515,000, and (2) by transfer of roughly $232,000 from available budget balances of other divisions of the Public Works Department.

Increased Metropolitan Assessments, together with a falling off of receipts from the sale of water, have been important contributory factors to the existence of an annual operating deficit. The falling off of receipts has been due somewhat to the business depression, but also to the ever increasing number of unpaid water bills which have mounted over the years to the enormous total of approximately $2,000,000.

In a city of the size of Boston it is perhaps astounding to discover that the Water Department is operating at a loss. With the turn of the year, the policy of the Water Department was changed in two respects: (1), the Water Department was to be conducted on a business basis, and (2) the Department was to be placed on a self-supporting basis.

Within the City of Boston three agencies are involved in the water system, namely, the Water Division, the Water Income Division and the Collecting Department. The Water Division had as its duty the maintenance and construction of the water system; the Water Income Division set and read meters and made out bills for water used; and the Collecting Department, as its name implies, collected all payments due on water bills, and notified the Income Division when bills were paid some two or three weeks after their payment. As a matter of practice, the consumer paid his water bills if he wished to. He let them slide if he cared to take a chance. If the bill for a quarterly period remained unpaid, the fact was not called to his attention on the next bill he received. There might be summonses, notices, etc., but no one paid much attention to them; and when the time came to pay the bill, the fees for summonses and demands were usually waived. The Income Division felt that with the

*Division Engineer, Department of Public Works, Boston, Mass.

reading of the meter and the rendering of the bill its duty was done and it was up to the Collector to do the collecting. Each department passed on duties to the other department. It is remarkable that the collections were as good as they were.

Reorganization. The system is to be revised; the Water Division is to be the Water Department. The Income Division is to be known as the Business Office of the Water Department, and the Collecting Department will still be the Collecting Department with the exception that the water customer's entire account will always be on hand in the Water Department office instead of in the Collector's office.

The stub plan of billing is to be put into effect, and the billing is to be performed on a billing machine. The water bill is to be made up of three parts; the body of the bill and two stubs. The main part of the bill will contain the name, address and code number of the customer. The bill will be dated and will carry the previous and current meter reading, cubic feet of water used in a quarter with amount of money to be paid for the period involved. It will also carry the amount of all unpaid bills and the total amount of money owed the department. Each of the two stubs will carry all of the information given on the main part of the bill, except the previous and current meter reading. The stubs, therefore, will show the current charges and unpaid water bills with the total amount due.

After the bills are made out, one stub is to be removed from the bill and filed away in the Business office, the remaining two parts of bill are to go to customer. When the bill is presented for payment the Collector will return the receipted main body of the bill to the customer and retain the stub to be forwarded to the business office after entering the name and amount on the Collector's blotter or ledger. The two stubs now in the business office will be matched together and filed away so that all stubs not matched together in the business office will be before the billing supervisors for drastic action. Meter readings are to be made on blank cards rather than loose leaf books. This arrangement is expected to prevent the meter readers from sitting at home or in the parks and making current readings from previous readings already in their possession.

Collections. A drive to collect the outstanding sum of $2,000,000 is being conducted with no undue severity. If the accumulated indebtedness is heavy, installments are accepted on proof of actual inability to pay. Due notice is given of curtailment or stoppage of water. Water to domestic consumers is never shut off completely. It is merely curtailed, or as we say, "reduce the flow." The supply of water is reduced to a trickle which causes inconvenience but which does not endanger health. This is not the first time that the City has

reduced the flow to domestic consumers. The practice has been followed on a small scale for the past ten years. As regards business places, the flow is to be stopped entirely and part payments are not encouraged.

When a customer has exhausted his credit with us a notice is sent by messenger or registered mail to the owner or occupant, giving the amount of the indebtedness and stating that the flow of water will be substantially reduced or entirely shut-off on a given date, 48 to 72 hr. from the time that the notice is received. The flow reduction notice reads as follows:—

PUBLIC WORKS DEPARTMENT
WATER DIVISION
BOSTON, MASS.

TO THE OWNER OR OCCUPANT OF PROPERTY AT

.....

According to our records the present status of your account for water with the City of Boston is as follows:—

A tax or service bill of the City of Boston is an important matter and must be paid promptly. The City is not justified in discriminating between its citizens as to the payment of their bills. The Department has exercised leniency in the payment of these bills beyond what now appears to have been a reasonable point.

This letter informs you that, as these bills have not been paid, despite adequate notice which you had, the water at the premises noted above will be either shut off entirely or the flow substantially reduced to the minimum of health and sanitary requirements on

.....

Water will not be turned on again until the full amount has been received by the Collector. The Collector's office is open until 2 p. m., and the water will be turned on again that same day where payments are received before that hour.

We hope that you will realize that the revenues of the City of Boston must be protected, that the City must not discriminate between its citizens, and further, that you will feel that it is the part of good citizenship for every one to see that sums owing the City are paid when due, without building up additional expense by requiring notices and special attention. We feel sure that it is only because this matter has not been viewed by you in this light previously that these bills have not been paid and that we shall have your immediate and full cooperation.

Very truly yours,

DANIEL M. SULLIVAN,
Division Engineer.

Education. The water-consuming public is to be educated, the work being based on three assumptions: fairness to all water users; courtesy from Water Department employees; and the function or duties that the Department expects the meter to perform.

We argue that it is not fair to the customer who pays his water bill to even think of raising water rates to put the Department on a self-supporting basis when there is a large outstanding balance. We would be penalizing the good customer and causing him to pay more money for water, while the non-paying customer received his water free.

We are educating our employees how to meet the customer—what to say and what not to say—not to assume an antagonistic attitude, even though the customer is about ready to fight.

We are attempting to educate the consumer to the fact that the meter is a registering device which performs the duty of recording the water that goes through the pipe. We say we are not interested in where the water goes, nor in what you do with it, but that once it

goes through the meter the Department will expect payment for all the water recorded.

"That never was done before!" This phrase is used continually by water customers who seek abatement of a water bill. Whenever a meter registered an unusually large amount of water because of an underground leak, for example, it was the practice in the past to have the customer present a statement from a plumber that a leak underground was repaired by him at the premises involved. Such a statement would be sufficient proof for the owner of a piece of property to have one-half of the excess over his normal bill abated. This practice has been stopped.

Customers would argue that during the winter months the water was allowed to run to waste to protect the meter from freezing during cold nights, and an abatement was usually allowed. This practice has been stopped. In fact, we tell the customer that a meter was set on the water pipe to record the amount of water used and that whatever the meter records will have to be paid for. This never was done before.

Results. Figures will speak for themselves. We are having considerable trouble in waiting on customers who are seeking their water bills, and the paying tellers windows remind one of the State Income Tax windows just before March 1.

Payments for the first two months of 1938 average $6000 per day; on March 1, the receipts for the day were $8,030; and on March 7, they increased to a maximum of $19,727. On March 12 the total amount collected was $136,450, as compared with $89,342 for the same period in 1937.

After the new system becomes established, there is no reason why the water department should not have an even better record of collection than the gas or electric light companies.

Summary of Discussion

Replying to questions by Messrs. Roger Esty, Elwood Bean, George Merril and Warren Scott, Mr. Sullivan stated that flow was reduced by partial closure of the curb cock; that this would not endanger household boilers; that there was 48- to 72-hr. notice in which to care for power boilers; that the fire comissioner and the insurance company were notified when sprinkler systems are shut off; that the health commissioner is notified when large apartment blocks are affected; and that the premises are inspected to see that water reaches at least the first floor at reduced flow.

NEW DEVELOPMENTS IN FLUID METERS THAT MAY AFFECT WATER METER DESIGN

BY WALTER PARKER*

[*Read September 23, 1937.*]

Water being a necessity for maintenance of life, there have been few communities in ancient or modern times that have not faced the problem of obtaining a continuous supply of potable water sufficient for the needs of the people. There are few places where good water is obtainable in large quantities, and in all densely populated centers tremendous sums must be spent to provide sufficient water for modern human needs. In ancient times large cities had to depend on the transportation of water in goat skins and earthern jars, which served both as containers for transportation and storage. There are a few places today where this primitive method of water supply is still in use, where the price of water is extremely high, and the quality is low. In ancient times ability to obtain water may have meant the rise or fall of a community or nation. Over two thousand years ago in Rome and Northern Africa the storage and distribution of large quantities of water were first developed in a scientific manner. The water supply of Rome is said to have supplied fifty gallons daily per capita, but after the downfall of the Roman Empire water systems were neglected and new construction was abandoned to such an extent that fourteen hundred years later the city of Paris was able to supply only two and one-half quarts per day per capita.

The value of a supply of pure water has been demonstrated quite recently to many of our large cities in the Ohio Valley, where flood waters polluted the water distribution systems and stopped the pumps. The inhabitants were surrounded by muddy, impure water and unable to obtain safe drinking water. When failure of water supply cuts off both drinking water and toilet facilities, people are brought very suddenly to appreciate the importance of water supply.

In England water service is considered a necessity of life which should be paid for on the basis of ability to pay rather than amount consumed, so that very few domestic services are metered. Compared to some large areas in America, water is more plentiful, but nevertheless the cost of pumping, filtering, and distributing it is high enough to warrant prevention of waste. The American opinion seems to be that, while water is a small item of expense, the consumer with limited

*Departmental Manager, Pittsburgh Equitable Meter Co., Pittsburgh, Pa.

ability to pay can and probably will waste just as much as anyone else unless a meter is provided to check waste. In America we have some large communities with an abundant supply of water available at high elevations, so that the installation of a distribution system is almost the entire requirement for the supply of good water. On the other hand, we have many more communities which have grown so rapidly that it has been necessary to extend the water system far away to reach new sources, and it has been necessary to buy large water sheds to protect the purity of the supply.

The city of New York is preparing to spend several hundred million dollars to provide water sheds, dams, pipe lines, and pumping stations for its growing population and for its industrial needs. Several cities on the Pacific Coast have invested hundreds of millions of dollars in water storage and supply systems, and in many cities the largest investment for any single utility purpose is in the water system.

It has been shown many times that in any city in which the use of water is not checked by meters, very close to half of the water is wasted. This statement sounds like propaganda for the use of water meters, but there are many cities that will furnish figures proving that after the complete installation of water meters the pumping requirements have been reduced by about fifty per cent.

Any such saving in the amount of water pumped has a tremendous bearing on the expense of water and consequently on tax rates. For example, the size of the pumping units, the investment in water lines, the chemical treatment costs, filtering beds, and the size and extent of water storage are nearly cut in two, so that both the investment and the cost of operation are cut approximately in half. Stated in another way, any unmetered community can double the capacity of its water supply system by installing meters. In addition to direct savings, there is the item of sewage disposal. The sewerage system must be much larger if it handles wasted water, and eventually sewage must be treated. This country cannot afford the risks to public health incident to dumping sewage into rivers and lakes or even into the ocean without treatment. It is an obvious fact that where water is wastefully discharged into the sewers, the cost of treating sewage is proportionately increased.

It is difficult to see how anyone can argue against the value of meters in reducing taxes, because they affect both the distribution of water and the disposal of sewage. In many communities where there are no meters, people let water run continuously in summer so that the water will be cold and in winter during cold periods so that it will not freeze in the pipes. Leaky water fixtures will frequently

waste two to three times as much water as is normally required by the occupants of a residence.

We are at present concerned with the measurement of water, probably the cheapest commodity on earth, delivered to the consumer during twenty-four hours of the day every day of the year. It is much cheaper than dirt. When we stop to think that pure water is on tap every minute at an average price of twenty cents per thousand gallons, we may realize how cheap it is. We obtain a still better picture when we reduce the thousand gallons to tons and consider weight delivered, which shows that over a ton weight of water is delivered for five cents. That is cheap freight service, with the water thrown in. When we go a step further and consider that in order to deliver the ton of water, if pumped at fifty pounds pressure, six horse power of energy has been expended, the price of five cents a ton delivered seems ridiculously cheap. Very few consumers realize how small is the price they pay for water.

Perhaps the earliest attempt to measure water in large volume was made by the Romans, who provided openings along their conduits which could be restricted to prevent upstream conduits being emptied before reaching the downstream sections. Charges for water to consumers were probably based on the size of orifice, and such charges correspond to the "flat rate" which is used in some localities in modern times.

The first recorded investigation in actual water measurement was done in 1797 by Venturi, an Italian engineer, but, although he conducted some interesting experiments in hydraulics to show the behavior of fluids in conduits, he did not develop a meter. It was not until 1887 that Clemens Herschel designed a meter in this country based on Venturi's experiments and named it a Venturi meter in his honor. Venturi meters are now available with indicating and totalizing mechanisms which cover a broad field of water measurement particularly for large volumes.

Domestic water meters were first produced in America in 1866, but it was not until 1890 that the annual production reached five figures. Since 1890 the industry has grown very rapidly, and it is estimated that there are seven million meters in actual use in this country today. Many of the original models or designs have been abandoned as the result of the most effective test: performance in actual service over long periods. The great majority of domestic meters in use in this country today are of the nutating disc type.

In view of the large volumes of water used per capita, even though it is so cheap, measurement naturally should be made as accurately as is economically possible, for we get into the multiplication

of high units by low unit costs, to be sure but still making sizeable totals, particularly where waste or carelessness prevails.

Water meters must be manufactured and sold at fair and reasonable prices; they must be reasonably accurate; they must operate reliably—that is, without sticking or stopping; their maintenance costs over a period of years must be within reason.

During the past five years or so, several manufacturers have made meters for gasoline measurement, normally accurate to within 0.1%, when measuring, for example, from one to fifteen gallons per minute. The meters cost more than do the same size of meters for water, and their maintenance cost is higher. This extra cost is justified by the fact that gasoline normally sells for about twenty cents a gallon instead of twenty cents per thousand gallons.

Special meters have been made for government service, metering beer in breweries for tax purposes, with a tolerance established at 0.2%. The Federal tax alone on the beer is five dollars per barrel of thirty-one gallons. In one large brewery a single meter measured one million barrels of beer in fifteen months for a tax of five million dollars. The same meter measuring the same volume of water might have collected, at water prices, six thousand dollars instead of five million dollars without computing the value of the beer.

Having obtained accuracies such as just mentioned for gasoline and beer, the question arises as to why this has not been done for water.

The first consideration is initial cost, which would probably be more than doubled. The second is that parts would have to be fitted with smaller clearances. This would probably necessitate the elimination of hard-rubber measuring elements, because, due to the difference of expansion of rubber and bronze—the coefficient of expansion of rubber being about four times that of the metal—close clearances will cause sticking or stoppage of meters with changes in temperature. Foreign particles also cause stoppages and would produce extra drag, higher absorptions, and consequent loss of accuracy. Perhaps the greatest source of annoyance and expense would be the shutting off of practically all water delivery if a tightly fitted meter stuck. Present type meters are far less subject to sticking, but if and when they do stick, the meter will pass enough water to keep the home going at a reduced rate. This is highly desirable in a water meter. Closely fitted meters, shutting off all but a dribble and being far more subject to sticking, might necessitate a twenty-four hour service, something on the order of a miniature fire department. It is also easy to imagine that if the service department, hurriedly installing a replacement meter, broke loose scale or hard deposits in the inlet pipe to the meter while making

up the joints, the service meter might stick before the service men returned to their headquarters.

Assuming that it would be necessary to eliminate hard rubber from very closely fitted meters, we would eliminate the material that is best lubricated by water and substitute a metal which, due to its extra weight and inferior water lubrication quality, would wear much faster and so reduce the life of the meter. From this we conclude that the closely fitted metal meter, even though more accurate when new, might at the end of a five-year period be far less accurate than present popular meters, so that the average accuracy over the five-year period might even be less than that obtained under present conditions.

When we consider the number of manufacturers of water meters in this country and in foreign countries making various types of meters for water service, we find that accuracies obtained with new meters come fairly within a reasonable degree of accuracy. It is safe to state that no commercially used domestic meter, in respect to accuracy, is so outstanding as to be within 0.2% accurate over a wide range. American water meters cannot claim closer than 1½% of accuracy over their range. The only reason for this is that it has not as yet been found economically practical to manufacture a meter of higher accuracy for water service.

In these times of rapid and surprising progress no one would say that "it can't be done". When one thinks of all the things that "couldn't be done" fifty years ago but that are now commonplace, one hesitates before using that combination of words. We have merely pointed out the obstacles that must be overcome.

Of course we may be able to improve the materials in water meters. Advances in metallurgy may result in our having a superior alloy from the standpoint of corrosion resistance at reasonable cost. In chemistry, the development of new synthetic plastics is a wide open field at the present time. A synthetic plastic that is heat resisting, of suitable strength, lubricated by water, with a satisfactory low thermal coefficient of expansion, and that does not impart a taste to water will find an important place in water meters.

Much progress toward greater accuracy has been made during recent years. Better machines have resulted in more accurate machining; better control of materials and, of course, better care of meters by water departments have resulted in longer life and higher percentage of accuracy over long periods. We must not lose sight of the fact that a water meter is a piece of machinery that runs with a poor lubricant, must be able to pass a certain amount of abrasive foreign matter and normally receives comparatively little service over a period of years.

We predict that, if there is any great improvement in domestic

water meters, it will be brought about by a new design different from that of any meter now on the market. The meter will have to combine the low first cost and continuous operation of the nutating disc meter with some more positive method of measuring displacement. The average domestic meter produces a revenue of about $12 annually, and it is not advisable to seek greater accuracy by increasing cost or liability of stoppage, or to risk more frequent replacement either because of stoppage or for repairs.

NEW DEVELOPMENTS IN THE CHEMISTRY OF RUBBER AND LIKE SUBSTANCES IN RELATION TO WATER WORKS PRACTICE

BY E. A. HAUSER*

[*Read January 20, 1938*]

The use of rubber in waterworks practice has always been a comparatively limited one, since it was primarily restricted to certain types of gaskets and valve seats, and the like, used in the pumping installations and in some pipe joints. However, with the increasing application of water disinfection by chlorination processes, new cases came into practice, an example being the application of rubber hoses to the transportation of chlorine. Yet, here, the fact that normal rubber is comparatively severely attacked by chlorine prevented any successful development until quite recently when there became available new synthetic organic products with rubber-like properties which are characterized by a very high resistence to chlorine and other excessively corrosive chemicals.

One such known substance is "Neoprene," chemically known as the polymerization product of 2 Chloro—3 Butadiene which is being used for the lining of rubber pipe or hose in the transportation mains of corrosive chemicals, like chlorine and others. Its high resistance to the influence of oils and similar solvents has made "Neoprene" also an excellent material for gaskets and rubber articles that frequently come in contact with oil, etc. Its age-resisting qualities, furthermore, make its use advisable wherever rubber parts are exposed to weather over prolonged periods of time.

In the jointing of bell and spigot pipes, the introduction of another rubber-like substance into the old customary sulphur jointing compounds has very materially improved their applicability and efficiency. This rubber-like synthetic material is "Thiokol". Chemically, "Thiokol" is polyethylene tetrasulphide. It imparts to the jointing compound immediate sealing properties, great resistance to mechanical and thermal shocks, unusual plasticity, a very low coefficient of expansion, etc.

Another development that uses rubber extensively is in the jointing of Transite pressure pipes, where a new special coupling known as the "Simplex" coupling is being applied. High-grade rubber rings are being used as water-tight joints in this type of water-

*Department of Chemical Engineering, Massachusetts Institute of Technology.

pipe and sewage-pipeline construction. Transite pressure pipe is characterized by being made of a material (asbestos-cement) that is not corroded by water and that, due to its comparatively smooth inner surface, prevents the formation of scale deposits, thus guaranteeing uniform carrying capacity and low pumping costs.

This development leads into another field, namely, that of the increasing application of synthetic resins as basic material for the manufacture of pipe-lines and the like. It is primarily the group of polyvinyl resins that are meant. Their high shock-resistance properties, low rate of expansion, and their faultlessly smooth surfaces, permitting the possibility of making any desired shape and bend and joint, constitute qualities that are ideally suited to such purposes. Outstanding in this field is the synthetic vinyl derivative mixed polymer known under the trademark "Mipolam."

Our increasing knowledge in the field of modern plastics and synthetic organic products is bound to reflect also on the future development of waterworks equipment and installations. Those in charge of such vital installations who are interested in keeping pace with new developments should not overlook the possibilities that are at present being offered, and will be offered in increasing amount, from now on, in their field of interest.

PROCEEDINGS

March Meeting

Hotel Statler, Boston, Mass.

Thursday, March 17, 1938.

President Warren J. Scott in the chair.

President Scott announced the death of James A. McMurray, Division Engineer, Water Income Department, Public Works Department, Boston, Mass.

Secretary Gifford announced the following elections to membership: *Members:* Leander G. Smith, Superintendent and Assistant Treasurer, Water District, Kennebunk, Me.; William F. Covil, Civil Engineer, Lexington, Mass.; Everett O. Boothby, Assistant Superintendent, Portland Water District, Portland, Me.; Ralph P. Hall (of Ralph P. Hall Co.), of Boston, Mass.; Laban C. Morrill, Water Works Engineer, Stoughton, Mass. *Associate:* Ralph P. Hall Company, pumping and power equipment, Boston, Mass.

The following resolution was presented and carried unanimously:

"WHEREAS the enactment of House Bill 872, now before the Legislature of Massachusetts, apparently removes from the Department of Public Health and the Water Works Commissioners charged with maintaining the purity of the public water supplies the power to prohibit fishing in ponds and reservoirs used as a source of public water supply; and

WHEREAS it is the opinion of the New England Water Works Association that in the interest of public health this power should remain with the said department and such water officials;

THEREFORE, be it resolved that the New England Water Works Association, in regular meeting assembled, March 17, 1938, notify the legislative Committee on Water Supply of its opposition to the passage of this proposed legislation."

A paper, "Concrete Standpipes," was read by Charles W. Sherman of Metcalf & Eddy, Consulting Engineers, Boston, Mass.

A paper, "Enforcing Payment of Water Bills," was read by Daniel M. Sullivan, Deputy Commissioner of Public Works, Boston, Mass. This subject was discussed by Roger W. Esty, Elwood L. Bean, George S. Merrill, Alexander H. O'Brien, Frank J. Gifford, Stephen H. Taylor, Richard D. Chase, A. O. Doane, Warren J. Scott, Harold L. Brigham, and Theodore L. Bristol.

A paper, "The Diesel Pumping Plant of the Milford Water Company," was read by M. F. Goff, Assistant Superintendent, Mil-

ford Water Company, Milford, Mass. Richard D. Chase, A. O. Doane, A. M. Symonds, Percy R. Sanders, and George F. Merrill took part in the discussion.

A sound motion picture, "Modern Magic in Fire Protection," was presented by the Rockwood Sprinkler Company, Worcester, Mass.

The President was empowered to appoint a Nominating Committee.

April Meeting

Endee Inn, Bristol, Conn.

Thursday, April 14, 1938.

President Warren J. Scott in the chair.

The President announced the following elections to membership: Roland F. Kittredge, Operator, Water Purification Plant, Willimantic, Conn.; Albert E. Casey, Superintendent of Public Works, Stoneham, Mass.

The meeting was welcomed to Bristol by Honorable Joseph W. Harding, Mayor of the City of Bristol.

A paper, "Public Relations," was read by Frederick G. Hughes, Executive General Manager, New Departure Manufacturing Company, Bristol, Conn.

A paper, "Bristol's New Filtration Plant," was read by James A. Newlands, Consulting Engineer, Hartford, Conn. Harold C. Chandler and President Scott took part in the discussion.

A paper, "Connecticut's New Plumbing Regulations as Affecting Supplies," was read by Leslie K. Sherman, Assistant Sanitary Engineer, Connecticut State Department of Health, Hartford, Conn. The discussion was opened by Stephen H. Taylor and participated in by President Scott, Francis H. Kingsbury, E. Sherman Chase, Harold L. Brigham, Arthur L. Shaw, George F. Merrill and Linn H. Enslow.

May Meeting

Hotel Brooks, Brattleboro, Vermont

Thursday, May 12, 1938.

President Warren J. Scott in the chair.

The Association was welcomed to Brattleboro by Honorable Martin A. Austin, Selectman of Brattleboro. Mr. Arthur C. King, Chairman of the Legislative Committee, announced pending legislation.

Mr. G. E. Hawkins, Superintendent, Water Department, Brattleboro, Vt., gave a talk on "Guniting Four Million Gallon Compensating

Reservoir." Roger W. Esty, E. Sherman Chase, H. L. Clark, Percy R. Sanders, and Raymond E. Snow took part in the discussion.

A paper, "Maintenance of a Distribution System in Northern New England," was read by James A. Sweeney, Assistant Superintendent, Manchester Water Department, Manchester, N. H. Joseph E. Moore, G. E. Hawkins, H. I. Clark, John P. Miller, George F. Merrill, and Henry T. Gidley took part in the discussion.

A paper, "Experiences in Watershed Sanitation," was read by Marshall S. Wellington, Sanitary Engineer, New Haven Water Company, New Haven, Conn. C. P. Moat, Francis H. Kingsbury, E. Sherman Chase, Leonard W. Trager, Percy A. Shaw, and Frederick O. A. Almquist took part in the discussion.

INDEX OF ADVERTISERS

	Page
Atlas Mineral Products Co.	xxv
Bailey, Howard E.	iii
Barbour, Frank A.	ii
Barrows, H. K.	ii
Bingham & Taylor	iv
Brainard, F. S. & Co.	xxv
Caldwell, Geo. A., Co.	xiii
Cement Lined Pipe Co.	xxv
Crosby, Irving B.	iii
Darling Valve & Mfg. Co.	ix
Donaldson Iron Co.	xxiv
Dresser, S. R., Mfg. Co.	Following front cover
Eddy Valve Co.	xv
Edson Corp., The	xxii
Ellsworth, Samuel M.	ii
Everett, Chester M.	ii
Fairbanks-Morse Co.	xxiv
Fay, Spofford & Thorndike	ii
Ford Meter Box Co.	xvii
Goodnough, X. Henry, Inc.	ii
Hayes Pump and Machinery Co.	xvii
Hays Mfg. Co.	xviii
Hersey Mfg. Co.	v
Hydraulic Development Corp.	x
Industrial Chemical Sales Division—West Virginia Pulp and Paper Co.	xix
Johns-Manville	xi
Kennedy Valve Mfg. Co.	xiv
Layne-Bowler New England Co.	iii
Leadite Co.	Back cover
Lock Joint Pipe Co.	Facing front cover
Ludlow Valve Mfg. Co.	xvii
Metcalf & Eddy	ii
Mueller Co.	xxxi
National-Boston Lead Co.	xvi
National Gunite Contracting Co.	iii
National Meter Co.	xii
National Water Main Cleaning Co.	viii
Neptune Meter Co.	vi
New England Water Works Association	viii
Pirnie, Malcolm	ii
Pierce-Perry Co.	xxv
Pitometer Company	ii
Pittsburg Equitable Meter Co.	Following front cover
Red Hed Mfg. Co.	xxxii
Rensselaer Valve Co.	xxiii
Reppucci, C. & Sons, Inc.	iii
Ross, Anthony & Son, Inc.	iii
Ross Valve Mfg. Co.	viii
Smith, The A. P., Mfg. Co.	xvii
Thorpe, Lewis D.	ii
Tredennick, J. H., Inc.	xxv
Turbine Equipment Co.	xx
U. S. Pipe and Foundry Co.	Facing back cover
Wailes Dove-Hermiston Corp.	xxiii
Wallace & Tiernan	xxi
Warren Foundry and Pipe Corp.	xxii
Weston & Sampson	iii
Whitman & Howard	iii
Wood, R. D., Co.	xxvi
Worthington Pump and Machinery Corp.	vii

(Classified index on page xxvii)

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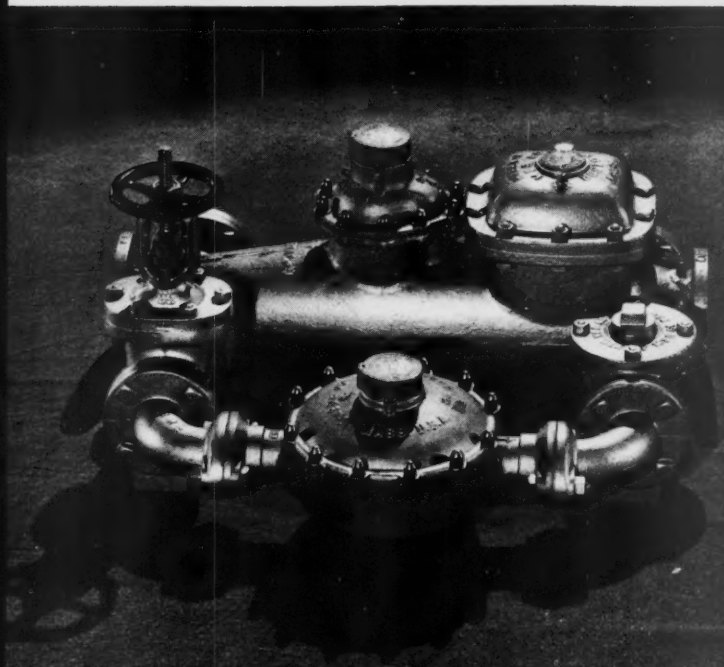

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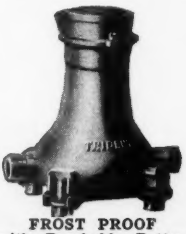

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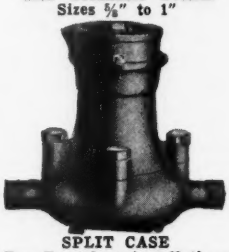

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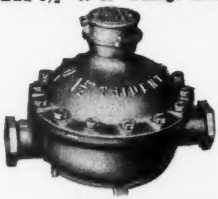

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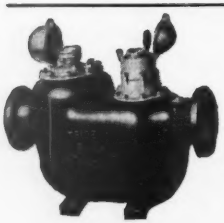

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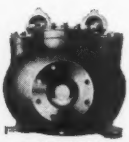

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WHEN looking over your requirements, remember that thousands of municipalities are getting satisfactory low-cost service from Worthington-Gamon Meters.

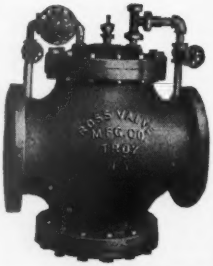
WORTHINGTON-GAMON METER COMPANY

General Offices: HARRISON, NEW JERSEY

Offices and Representatives in Principal Cities

WORTHINGTON-GAMON

WG8-7



THE STANDARD FOR OVER 50 YEARS

Automatic Pressure Control Valves
Water Works Specialties

Altitude — Pressure Reducing — Surge Relief and Combination Valves
Portable Fire Hydrants
Hydraulic Booster Pumps

ROSS VALVE MFG. CO., Inc. Troy, N.Y.

Health and Protection First

Water delivered through dirty pipes may be a MENACE.

Incrusted water pipes mean inefficiency and loss of Fire Protection.

We Guarantee the Results of Our Method of Cleaning.

WRITE US.

National Water Main Cleaning Co.

50 Church Street **New York City**

THE NEW ENGLAND WATER WORKS ASSOCIATION

is very desirous of obtaining
the following early issues of the Journal:

Vol. 2, Nos. 1, 2, 3.	Vol. 8, No. 1.
Vol. 3, No. 1.	Vol. 11, No. 4.

Please communicate with the office
613 Statler Building
Boston, Mass.

YOU'RE THE DOCTOR

For healthy water supply and sewage systems . . . prescribe Darling Valves and Fire Hydrants!

Your choice of valves and fire hydrants can make a big difference in the health and well-being of the entire system. Choose the right valve and hydrant—and you will avoid plenty of trouble!

Darling Valves and Fire Hydrants are designed and built to do a superlative job. Darling users will tell you that Darlings *save money and worry*—every day.

Darling representatives have the experience and skill to solve your particular problems. Get in touch with our nearest representative today.

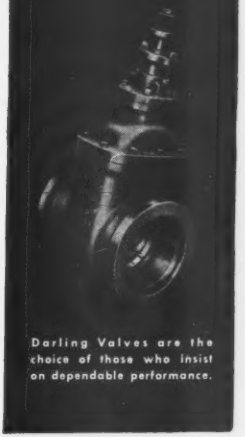
Darling Valve & Manufacturing Co.
Williamsport, Pa.

Representatives in

New York
Philadelphia
Pittsburgh
Houston
Oklahoma City
Toledo
Evanston, Ill.

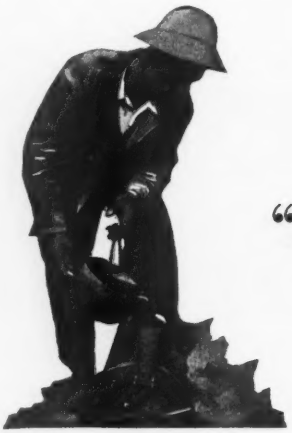
The large barrel of the Darling Fire Hydrant assures ample water supply at all times.
Experience favors Darling Valves.
For sewage disposal and filtration plants, Darling Gate Valves are different in operation—more dependable in performance.

The large barrel of the Darling Fire Hydrant assures an abundance of water at all times. There are five other important advantages.



Darling Valves are the choice of those who insist on dependable performance.

DARLING
GATE VALVES & FIRE HYDRANTS



"Never a Failure in Over 22 Years!"

WE like to talk about saving in material and labor—the strong, tight, flexible joints made with Hydro-Tite—the thousands of miles of pipe jointed with it.

More important, perhaps, is the fact that with hundreds of users—each laying pipe under different conditions—not a single one has reported a Hydro-Tite failure.

We make it very easy for you to change over to Hydro-Tite. Our Data Book outlines a novel proposition. Write for it.

A Symbol of Quality

A Dependable Self-Caulking Joint Compound

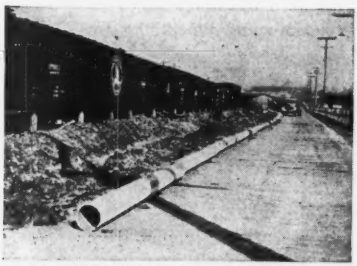
HYDRO-TITE

HYDRAULIC DEVELOPMENT CORPORATION

Main Sales Office: **50 Church Street, New York, N. Y.**
General Offices and Works: **West Medford Station, Boston, Mass.**

MORE MILES of TRANSITE PIPE...

OHIO: Hundreds of American municipalities have made sizable savings in water-distribution costs by using J-M Transite Pipe. Shown above is one of many complete Transite systems installed in 1937.



WASHINGTON: Simplex Couplings stay tight even when subjected to vibration from near-by railroad trains. And Transite's immunity to electrolysis, when laid near electric lines, is added assurance of virtually trouble-free life.

MAINE: Because Transite Pipe is non-metallic, tuberculation is a physical impossibility. A permanently smooth interior keeps pumping costs low, delivery capacity high, indefinitely.

and every foot CUTS COSTS

TROUBLEPROOF pipe means real economy on water-transportation systems. And on hundreds of installations all over the country, J-M Transite Pressure Pipe is bringing operating costs to new lows.

Transite Pipe, made of asbestos fiber and cement, is non-metallic and highly resistant to corrosive soils and ground waters. Tuberculation is impossible. High delivery capacity continues indefinitely . . . keeping pumping costs down.

At installation, too, Transite Pipe provides new economy. Unskilled workmen can lay it rapidly. Simplex Couplings help speed up assembly . . . eliminate caulking operations. Trenches can be held to minimum widths. The result: substantial savings in time, labor and materials.

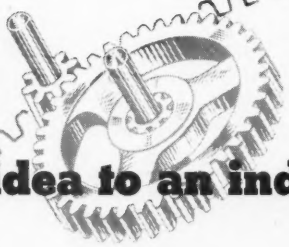
Yet Transite Pipe costs no more than ordinary pipe. Check complete details; you'll see other ways you can benefit. Write for the new Transite Pressure Pipe brochure. Johns-Manville, 22 East 40th St., New York, N. Y.

JM Johns-Manville

TRANSITE PRESSURE PIPE

An Asbestos Product

PROGRESS



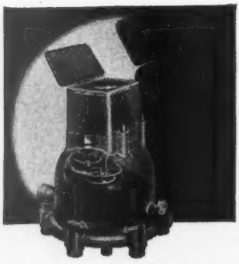
from an idea to an industry

Throughout the years that have seen the greatest progress in water works engineering, the National Meter Company has led in developments which have advanced metering from an experiment to an economic essential . . . developments which have advanced the manufacture of water meters from an idea to an industry.

The National Meter Company was first to introduce the velocity type of meter, first to offer the enclosed intermediate and the straight reading register, and first to develop and perfect frost protection features.

Then, too, both the positive-displacement oscillating piston and the disc-type meter were invented by Dr. L. H. Nash of the National Meter Company . . . meters so simple in design and construction, so accurate and dependable in operation that their basic principles are still used universally and are recognized as outstanding contributions to water works engineering.

The progress and leadership of the National Meter Company will be devoted in the future, as in the past, to the service of the water works industry.



NATIONAL METER COMPANY
4207 First Avenue • Brooklyn, N. Y.
BOSTON • CHICAGO • DALLAS
LOS ANGELES • SAN FRANCISCO

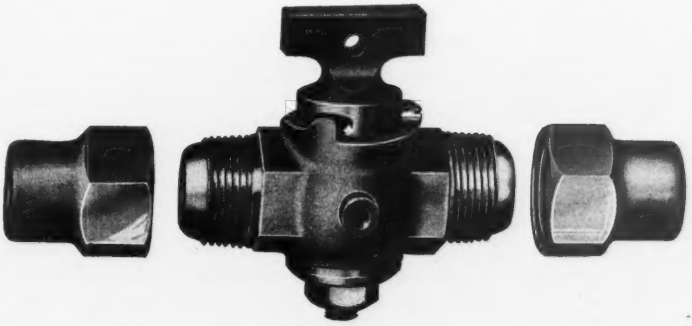
NATIONAL *Meters*

BUILDING ON QUALITY and SERVICE

Because we build QUALITY into our BRASS GOODS line; accurate mixture of the right metals, uniform construction, strength and pressure standards—and

Because we maintain adequate stocks and give prompt shipping SERVICE—and

Because our prices are right—every season closes with more new customers added to our long list of discriminating users



Illustrative of our QUALITY line, is the above A-13 copper to copper CURB Stop & Waste. Stocked in ¾″ and 1″ sizes.

NEWS. The new CALDWELL DRAINABLE GATE VALVES, double female also copper tube inlet, adopted in March, are making new customers constantly. Proven and endorsed through unsolicited repeat orders received.

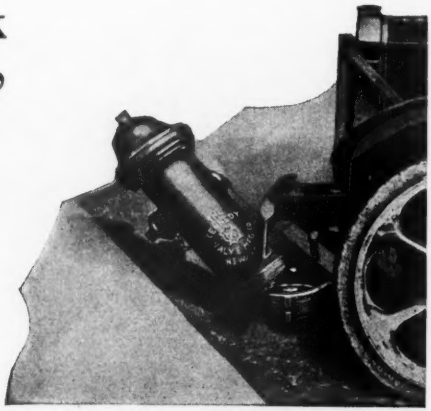
GEORGE A. CALDWELL CO.

Phone BLUe Hills 2791

BOSTON MATTAPAN SQUARE MASS.

When a truck runs "amuck"

and crashes into a Kennedy SAFETOP Fire Hydrant



Send for Bulletins

IN NO INSTANCE—either on demonstration test or in accidental collisions, has the Safety Breakable Section ever failed in its function of preventing breakage of the major and expensive parts of the SAFETOP. The only parts that need be replaced after breakage are a threaded standpipe ring and a stem coupling. These parts cost $6.00—a very small fraction of the cost of the complete hydrant which must ordinarily be replaced after breakage from accidental collision. Repairs do not necessitate shutting off the water pressure, and can be completed within half an hour.

In addition to its unique safety feature, the Kennedy SAFETOP Fire Hydrant assures quick response with full main pressure delivered at the nozzle in fire service; and its simple operating mechanism and wear-resisting valve facings last indefinitely. Write for bulletins describing its many special features.

The Kennedy Valve Mfg. Co., Elmira, N. Y.

KENNEDY
SAFETOP FIRE HYDRANT
REG. U.S. PAT. OFF.

EDDY

Valves and Hydrants

Distinctive Features of the EDDY Valves

ONLY three parts are moved by the Stem—the ball and the two gates. Each gate is hung from one of the trunnions on the ball. A convex surface at the base of each trunnion fits a concave surface on the back of the gate. This allows the gates to adjust themselves properly to their seats. . . . The gates, being free to revolve on the trunnions, do not always seat in the same position. Gates are center bearing and adjustable. They are forced to their seats with equal pressure at all points. There are two hooks on the ball on sizes 4-inch and larger, which loosely engage with the gates.

Distinctive Features of Our New Swivel Top Hydrants

THIS new hydrant has all the advantages of the popular EDDY fire hydrant, plus several additional features. . . . Nozzles are in a short flanged section of the standpipe, which may be unbolted and turned to different positions. . . . Should the standpipe be broken, only the casting below the swivel head need be replaced. . . . To raise the hydrant to conform to a new grading, simply insert a flanged extension piece below the swivelhead. No digging. To add a steamer nozzle at any time, it is only necessary to replace the swivel head with one having a steamer nozzle.

Besides valves and hydrants to meet the most exacting demands of waterworks service, the complete EDDY Line includes: valves for sewage disposal works; valves for steam, gas and oil; check valves, foot valves, plug valves, shear gates, indicator posts, etc. EDDY valves are made in three classes: Iron Body Bronze Mounted, All Bronze, and All Iron Valves.

Eddy Valve Co.

WATERFORD NEW YORK

LEAD PIPE

Used by leading WATER WORKS for over 50 years. If proper weight is used, combined with good workmanship, your FINAL COST is assured, as REPAIRS are unnecessary. LEAD PIPE is sufficiently ductile to conform to any sagging condition. There is no RUSTING of pipe or joints; no DISCOLORATION of water.

LEAD WOOL

Every Atom Pure Lead

For calking pipe joints under the most difficult conditions; for overhead joints or in wet places where the use of molten lead is not only impracticable but dangerous — LEAD WOOL may be used to advantage.

It makes an absolutely tight joint which will withstand the highest pressure, yet be sufficiently elastic to allow considerable sagging or settling of the pipe, without danger of leaks.

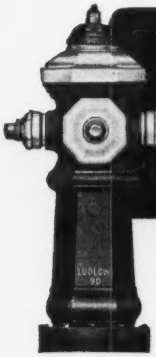
Compared with the poured joint, in this respect, the superiority of LEAD WOOL is apparent.

LEAD PIPE TIN-LINED LEAD PIPE
PURE BLOCK TIN PIPE SOLDER
DUTCH BOY WHITE LEAD & RED LEAD

Highest Quality Soft Virgin Omaha Pig Lead

National-Boston Lead Co.

800 Albany St. Boston, Mass.



Choose... LUDLOW HYDRANTS

When you choose hydrants, be sure they deliver full flow at all times —eliminate surge and water hammer —break off above the ground, and can be replaced without digging.

Ludlow Hydrants do all that, and combine graceful design with low maintenance costs as well. Send for free folder.

The LUDLOW VALVE MFG. CO.
TROY, NEW YORK

PUMPS { CENTRIFUGAL, STEAM, TRIPLEX, ROTARY

POWER { DIESEL, GASOLINE, STEAM, ELECTRIC

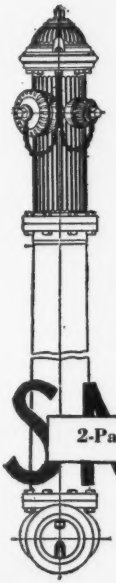
PARDEE CHLORINATORS { DIRECT, SOLUTION, VACUUM

PORTABLE { LABOUR SELF-PRIMING PUMP, AIR COMPRESSORS AND TOOLS

For Rent or Sale

HAYES
PUMP & MACHINERY CO.
125 Purchase Street Boston, Mass.
Hancock 7957--7958

Minimum Trouble and Expense When Breaks Come



THE Smith Two-Part Standpipe Hydrant was designed to make the expense as small as possible when there is a break as the result, for instance, of careless auto truck driving.

The upper section will leakage of water because always break first. It can easily be renewed. And of course there can be no the main valve is of the compression type.

SMITH
2-Part Standpipe Hydrant

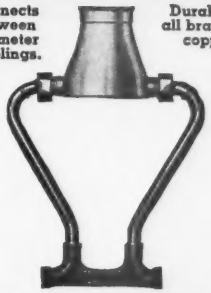
THE A. P. SMITH MFG. COMPANY
East Orange, N. J.

The Ford Resetter

AN IDEAL FITTING *for* RAISING TOO-LOW METERS EASILY

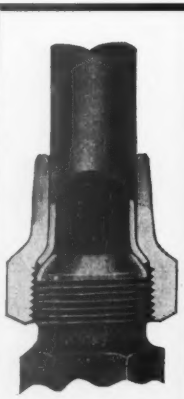
Connects between old meter couplings.

Durable,— all brass and copper.



Improve your old meter settings by installing the RESETTER. Bring your water meters up out of the mud and water so that they are easier to read and take care of. Write for catalog.

The Ford Meter Box Co.
WABASH, INDIANA



HAYS begins at the MAIN

Since 1869 Hays has built to the exacting requirements of the Water Works Industry. Tap the Main with a Hays Tapping Machine and follow through with dependable Hays Service Fittings for iron pipe, lead or copper. Hays Corporation Stops and Curb Stops are made in all styles and sizes. Write for Catalog.

HAYS RADIUS SEAL JOINT

This type of joint was designed to meet the specific requirements of certain Water Departments and is available directly on all styles of Hays Copper Service fittings.

HAYS DOUBLE SEAL JOINT

This joint features a 2-faced pipe flare (45° + 90°) and a 2-faced machined seal (45° + 90°) which is by test the strongest flanged connection available. Available by Adapter on any Corporation Stop and directly on more than 400 styles and sizes of Hays Service and Plumbing fittings.

USE THE HAYS COPPER SERVICE METHOD

Hays Adapts Copper to meet any Requirements

As a pioneer in the development of copper pipe services, Hays has popularized its use by making it easily adaptable to meet existing line equipment economically. Every community can have the advantage that copper pipe makes possible from Main to Curb to House with the Hays Copper Service Method. Let us have your problem.

HAYS MFG. CO.

ERIE, PA.

HAYS

PLUMBING BRASS
BATH FIXTURES
COPPER PLUMBING
GAS SERVICE GOODS
WATER SERVICE GOODS
IRON PRODUCTS
TAPPING MACHINES

JUST as these fishermen are going over their nets and floats for greater efficiency and catch, water plant operators should adjust their treatment to compensate for Summer changes in their water supply.

Warm weather invariably promotes excessive algae and other organic growths in raw water supplies—resulting in increased tastes and odors, with its attendant difficulties in controlling. At this season there is also the increased need of water for drinking and other purposes.

MAKE YOUR WATER SUPPLY INVITING TO DRINK . . . . MAKE IT PALATABLE . . . .
IT'S EASY TO DO WITH AQUA NUCHAR

INDUSTRIAL CHEMICAL SALES

DIVISION WEST VIRGINIA PULP AND PAPER COMPANY

230 Park Avenue
New York, N. Y.

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Cleveland, Ohio

Manufacturers of NUCHAR *Activated Carbon*

DE LAVAL

Pumps selected by the

Metropolitan District Commission

for latest pumping station

Two 125 H.P. motor-driven centrifugal pumps in the new Belmont, Mass., pumping station of the Metropolitan District Commission

Each pump is designed to handle 3 million gallons of water per day against a total dynamic head of 162 feet.

The complete switchboard shown in the photograph is of the Dead Front type. We furnished pumping equipment and switchboard all erected and wiring between switchboard and motors.

TURBINE EQUIPMENT COMPANY

OF NEW ENGLAND

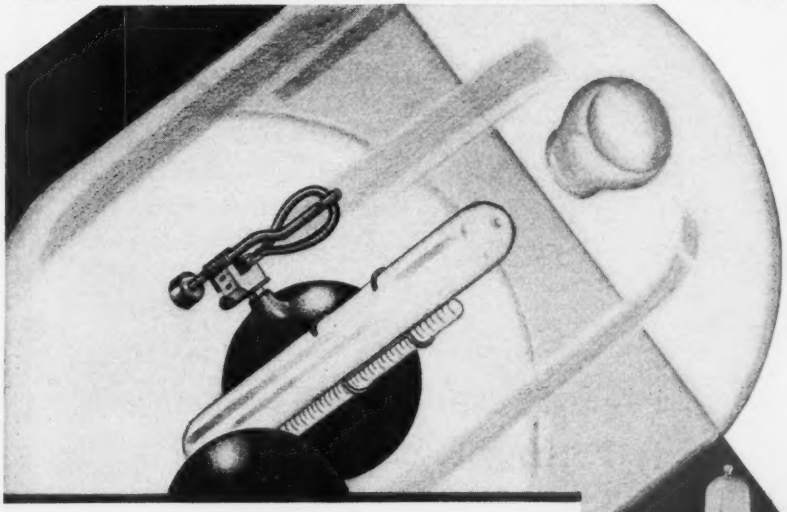
80 Federal Street **Phone LIBerty 5993-5994** **Boston, Mass.**

New England Representative for

DE LAVAL STEAM TURBINE COMPANY

TRENTON, N. J.

REAL VALUE FOR YOUR CHLORINATION DOLLAR



GLASS HOUSES

are indispensable to chlorination

TROUBLE in a Visible Vacuum Chlorinator doesn't have a chance to develop into anything serious because the working parts are under the glass bell jar, always in plain sight.

But this is only one reason for Visible Vacuum Control. Visibility has an important corollary—Accessibility. When adjustments are needed—just drain the tray, lift the bell jar and the complete control mechanism is accessible for cleaning, repair or adjustment.

This important principle of combining Visibility and Accessibility, reinforced by standardized, high quality manufacturing and careful selection of the best materials has made the name "A W&T Visible Vacuum Chlorinator" synonymous with dependability, accuracy and low maintenance expense for over fifteen years.

When you are faced with the selection of equipment for the control of chlorination, visit a W&T Visible Vacuum installation. There you will learn why the Engineer, Superintendent and Public Health Official alike agree that Visible Vacuum Control is a real assurance for full value on your Chlorination dollar. Ask for technical publications 38, 157 and 158 describing W&T Visible Vacuum Chlorinators.

WALLACE & TIERNAN CO. INC.

Manufacturers of Chlorine and Ammonia Control Apparatus

NEWARK, N. J. Branches in Principal Cities

"The Only Safe Water is a Sterilized Water"

CHLORINATORS FOR WATER WORKS • SEWAGE PLANTS • SWIMMING POOLS
INDUSTRIAL PLANTS

SA-23

EDSON
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Hand Pumps – Power Pumps
Truck or Trailer Mounted
Edson Special Suction Hose
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Strainers, Adapters, etc.

Distributors for
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Standard Hydrant Protector
Universal Water Leak Detector

THE EDSON CORPORATION
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New York: 142 Ashland Place, Brooklyn

Warren Foundry & Pipe Corp.
ALSO
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SALES OFFICES
11 BROADWAY, NEW YORK
75 FEDERAL STREET, BOSTON, MASS.
Manufacturers of
CAST IRON PIPE
Flanged Pipe ***Flexible Joint Pipe***
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Special Castings ***Short Body B. & S. Specials***
Warren (WF) Spun Centrifugally Cast Iron Pipe
WORKS: PHILLIPSBURG, N. J. and EVERETT, MASS.
Large Stock Enables Us to Make Prompt Shipments

Few Products In Any Field Are So Clearly Recognized for Superiority of Service As

SPUN BITUMASTIC LINING

REG. U.S. PAT. OFF.

The Report of the Committee on PIPE LINE COEFFICIENTS of the New England Water Works Association definitely proves the absolute economic necessity of lining all water pipe.

BITUMASTIC SPUN LINING is assigned a higher value of the coefficient *C* than any other lining. This insures the highest initial flow. More important —this high flow will be maintained for many years.

BITUMASTIC ENAMEL is not affected by tuberculating waters. Actual inspection of pipe in service and carrying tuberculating waters shows BITUMASTIC unaffected. This is real proof.

No taste, color, or odor imparted to the water.

Wailes Dove-Hermiston Corporation

General Office
17 Battery Place
New York

New England Representative
W. D. Barker, Room 606
250 Stuart St., Boston, Mass.

The "COREY" Hydrant

Why you should use GENUINE COREY HYDRANTS

Water to Nozzles quicker than other Hydrants.

Standpipe can be removed and replaced without shutting off water in main.

Ease of operation insured by knuckle joint principle.

Only 13 turns to completely open or completely close, but full flow with only 4 turns.

Nozzles screwed in — not leaded.

Can be lengthened without shutting off water.

Will not flood if broken.

Positive Drip — cannot freeze.

Standpipe can be revolved quarter turns — Nozzles always in right direction.

Ask for our Catalog "G"

CHARLES L. BROWN,
New England Sales Manager, Lock Box 2, Northboro, Mass.

RENSSELAER VALVE COMPANY
TROY, N. Y.

DONALDSON IRON CO.

MANUFACTURERS

. . . OF . . .

CAST IRON PIPE

. . . AND . . .

Special Castings for Water and Gas
Also Flange Pipe and Fittings

EMAUS, LEHIGH COUNTY, PA.

New York Office: E. A. NOONAN, 225 Broadway
New England Office: CHARLES F. GLAVIN, 35 Winslow Rd., Brookline, Mass.

WATER PROBLEM SOLVED BY NEEDHAM, MASS.

X. Henry Goodnough, Inc.
Engineers

● **The town of Needham can well be proud of its new water pumping station, for as city officials state, "Our water troubles will be over for a good many years to come." Believed to be the first Municipal Diesel driven Turbine Pump installation in New England, this system is equipped with a Fairbanks-Morse single-cylinder Diesel which powers an F-M Turbine Pump to economically supply the best quality water from an underground reservoir.**

Needham, as well as many other New England municipalities equipped by Fairbanks-Morse, is showing a substantial saving in operating costs. Get the facts on F-M power and pumping equipment by writing to Fairbanks, Morse & Co., 178 Atlantic Avenue, Boston, Mass.

FAIRBANKS-MORSE COMPANY

Power - Pumping - and - Weighing - Equipment

Water Works Officials

Cultivate the goodwill of your customers

Use the

METER-MASTER

To settle high bill complaints

The METER-MASTER is the only portable rate recorder that gives a continuous record for 6 and 24 hour periods, or 24 and 72 hour periods, from either a single or a compound meter.

F. S. BRAINARD & CO.
246 Palm Street
Hartford, Connecticut

Cement Lined Service Pipe

Cement lined pipe has eliminated corrosion and metal contamination for 60 years

Write for Literature

Cement Lined Pipe Co.
Lynn, Mass.

SPEED UP YOUR PIPE JOINTING

No need to wait days for leaks to heal. Made with Tegul-MINERALEAD, joints meet any reasonable leakage specification at once. Trenches can be backfilled and premises cleared up. ■ Joints are permanently tight. ■ The 10 lb. ingots are easy to ship, store and handle; impervious to water—can be stored outdoors. For booklet, address

A Tegul PRODUCT

Tegul-MINERALEAD

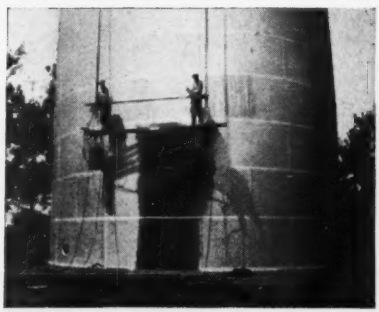
The ATLAS MINERAL Products Company of Pennsylvania
Mertztown . . . . . Pennsylvania

PIERCE - PERRY CO.

Wholesalers of

Water Works Brass Goods.
Byers Wrought Iron Pipe.
Youngstown Steel Pipe.
Valve and Service Boxes.

236 Congress St., Boston, Mass.
Telephone, Hancock 7817-7818



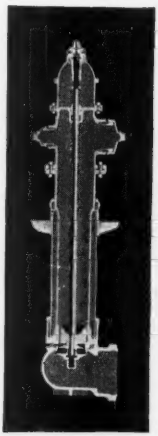
STANDPIPE PAINTING

Sandblasting Power Chipping
Power Wire Brushing

PAINTING by BRUSH or SPRAY

J. H. TREDENNICK, Inc.
10 High Street Boston, Mass.

BIG-MUSCLED FIREMAN Can't Hurt Mathews



● The Mathews operating stem is proof against the strongest fireman with the biggest wrench.

HE CAN'T TWIST THE STEM, for only the operating nut revolves. The stem itself moves straight up and down. It is free from torsion.

HE CAN'T BUCKLE THE STEM, for his force is never exerted along its whole length. A bronze stop nut meets the stuffing box as the valve comes full-open, ending further motion before the stem itself hits bottom.

The worst a fireman can do with any wrench is batter the wrought iron shield-nut which is instantly replaceable. One more proof that you'll never dig up a Mathews. Specify this hydrant by name.

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CLASSIFIED DIRECTORY OF ADVERTISEMENTS

ACTIVATED CARBON.
Industrial Chemical Sales Division—West Virginia Pulp and Paper Co. xix

AIR CONDITIONING EQUIPMENT.
Worthington Pump and Machinery Corp. vii

AIR COMPRESSORS.
Hayes Pump and Machinery Co. xvii
Worthington Pump and Machinery Corp. vii

ASBESTOS CEMENT PIPE.
Johns-Manville xi

BRASS GOODS.
(See also Pipe, Brass.)
Caldwell, George A., Co. xiii
Cement Lined Pipe Co. xxv
Hays Mfg. Co. xviii
Mueller Co. xxxi
Pierce-Perry Co. xxv
Red Hed Mfg. Co. xxxii
Smith Mfg. Co., The A. P. xvii

CALKING MACHINERY AND TOOLS.
Mueller Co. xxxi
Smith Mfg. Co., The A. P. xvii

CAST IRON PIPE. (See Pipe, Cast Iron.)

CEMENT LINED PIPE. (See Pipe, Cement Lined)

CHECK VALVES.
Darling Valve & Mfg. Co. ix
Mueller Co. xxxi

CHEMICAL FEED APPARATUS.
Wallace and Tiernan Co., Inc. xxi

CHLORINATORS.
Wallace & Tiernan Co., Inc. xxi

CLAMPS, RIVER.
Dresser, S. R., Mfg. Co. Following front cover

CLAMPS, SERVICE.
Mueller Co. xxxi

CLEANING WATER MAINS.
National Water Main Cleaning Co. viii

COCKS, CURB AND CORPORATION.
Caldwell, George A., Co. xiii
Hays Mfg. Co. xviii
Mueller Co. xxxi
Pierce-Perry Co. xxv
Red Hed Mfg. Co. xxxii
Smith Mfg. Co., The A. P. xvii

CONCRETE PIPE. (See Pipe, Concrete.)

CONDENSERS.
Hayes Pump and Machinery Co. xvii
Worthington Pump and Machinery Corp. vii

CONTRACTORS' EQUIPMENT
Worthington Pump and Machinery Corp. vii

CONSTRUCTION EQUIPMENT.
Edson Corp., The xxii

CONSTRUCTION AIR TOOLS
Worthington Pump and Machinery Corp. vii

CONTRACTORS.
Layne-Bowler New England Co. iii
National Water Main Cleaning Co. viii
Reppucci, C. & Sons, Inc. iii
Ross, Anthony & Son iii

COUPLINGS, FLEXIBLE PIPE.
Dresser, S. R., Mfg. Co. Following front cover

COUPLINGS, REPAIR.
Caldwell, George A. Co. xiii

CURB BOXES.
Bingham and Taylor iv
Caldwell, George A., & Co. xiii
Hays Mfg. Co. xviii
Mueller Co. xxxi

CURB AND VALVE BOX FINDERS.
Caldwell, George A., Co. xiii

DIAPHRAGMS, PUMP.
Edson Corp., The xxii

CLASSIFIED DIRECTORY OF ADVERTISEMENTS *(Continued)*

ENGINEERS. Page

Bailey, Howard E. ... iii
Barbour, Frank A. ... ii
Barrows, H. K. ... ii
Crosby, Irving B. ... ii
Ellsworth, Samuel M. ... ii
Everett, Chester M. ... ii
Fay, Spofford & Thorndike ... ii
Goodnough, Inc., X. Henry ... ii
Metcalf & Eddy ... ii
Pirnie, Malcolm ... ii
Pitometer Co. ... ..
Thorpe, Lewis D. ... ..
Weston & Sampson ... iii
Whitman & Howard ... iii

ENGINES. (See Pumps and Pumping Engines.)

EQUIPMENT. (See Contractors' Equipment.)

ERECTORS, WATER WORKS AND POWER MACHINERY

Hayes Pump and Machinery Co. ... xvii
Turbine Equipment Co. ... xx

FEED WATER HEATERS.

Hayes Pump and Machinery Co. ... xvi
Ross Valve Mfg. Co. ... v
Worthington Pump and Machinery Corp. ... vii

FILTER RATE CONTROLLERS AND GAGES. (See Rate Controllers)

FILTERS AND WATER SOFTENING PLANTS.

Ross Valve Mfg. Co. ... viii

FILTRATION PLANT EQUIPMENT.

Ross Valve Mfg. Co. ... vii
Wallace & Tiernan Co., Inc. ... xxi

FLEXIBLE JOINTS.

U. S. Pipe and Foundry Co. ... Facing back cover

FLAP VALVES.

Mueller Co. ... xxxi

FURNACES, ETC.

Atlas Mineral Products Co. ... xxv
Leadite Co., The ... Back cover
Mueller Co. ... xxxi
Smith Mfg. Co., The A. P. ... xvii

GATE VALVES. (See Valves.)

GUNITE.

National Gunite Contracting Co. ... iii

HOSE, SUCTION AND CONDUCTION.

Edson Corp., The ... xxii

HYDRANT PROTECTORS.

Edson Corp., The ... xxii

HYDRANTS, FIRE.

Darling Valve & Mfg. Co. ... ix
Eddy Valve Co. ... xv
Kennedy Valve Mfg. Co. ... xiv
Ludlow Valve Mfg. Co. ... xv
Mueller Co. ... xxvi
Rensselaer Valve Co. ... xxiii
Ross Valve Mfg. Co. ... v
Smith Mfg. Co., The A. P. ... xvii
Wood, R. D., Co. ... xxvi

HYDRANT PUMPS.

Edson Corp., The ... xxii

INSTRUMENTS. (See Water Works Instruments.)

LEAD.

National-Boston Lead Co. ... xvi

LEAD PIPE. (See Pipe, Lead.)

LEAD WOOL.

National-Boston Lead Co. ... xvi

LEAK FINDERS.

Pitometer Co. ... ii
Edson Corp., The ... xxii

CLASSIFIED DIRECTORY OF ADVERTISEMENTS *(Continued)*

METERS, WATER AND OIL. Page
Hersey Mfg. Co. v
National Meter Co. xii
Neptune Meter Co. vi
Pitometer Co. ii
Pittsburgh Equitable Meter Co. ..... Following front cover
Worthington Pump and Machinery Corp. vii

METER COUPLINGS.
Ford Meter Box Co. xvii

METERS (VENTURI TYPE).
Mueller Co. xxxi
National Meter Co. xii

METER BOXES.
Bingham and Taylor iv
Ford Meter Box Co. xvii

METER MASTER.
Brainard, F. S. & Co. xxv

METER TESTERS.
Ford Meter Box Co. xvii
Mueller Co. xxxi

OIL ENGINES, DIESEL.
Fairbanks-Morse Co. xxiv
Hayes Pump and Machinery Co. xvii
Worthington Pump and Machinery Corp. vii

PAINTING.
Tredennick, J. H., Inc. xxv

PIPE, BRASS.
Caldwell, George A. Co. xiii
Pierce-Perry Co. xxv

PIPE, CAST IRON (AND FITTINGS).
Donaldson Iron Co. xxiv
U. S. Cast Iron Pipe and Foundry Co. ..... Facing back cover
Warren Foundry and Pipe Corp. xxii
Wood, R. D., Co. xxvi

PIPE, CEMENT LINED.
Cement Lined Pipe Co. xxv

PIPE, CONCRETE.
Lock Joint Pipe Co. ..... Facing front cover

PIPE CUTTING MACHINES.
Smith Mfg. Co., The A. P. xvii

PIPE JOINTING MATERIAL.
Atlas Mineral Products Co. xxv
Hydraulic Development Co. x
Leadite Co., The ..... Back cover

PIPE, LEAD.
National-Boston Lead Co. xvi

PIPE LINING.
Wailes Dove-Hermiston xxiii

PIPE, TRANSITE.
Johns-Manville xi

PIPE, WROUGHT IRON AND STEEL.
Pierce-Perry Co. xxv

PLUG VALVES.
Mueller Co. xxxi

PITOMETERS.
Pitometer Co. ii

PORTABLE AIR COMPRESSORS. (See Air Compressors)

POWER WIRE BRUSH, INC.
Tredennick, J. H., Inc. xxv

PRESSURE REGULATORS.
Caldwell, George A., Co. xiii
Mueller Co. xxxi
Ross Valve Mfg. Co. viii

CLASSIFIED DIRECTORY OF ADVERTISEMENTS *(Continued)*

Page

PROVERS, WATER.
Pittsburgh Equitable Meter Co. ..... Following front cover

PUMPS AND PUMPING ENGINES.
Edson Corp., The xxii
Fairbanks-Morse & Co. xxiv
Hayes Pump and Machinery Co. xvii
Ross Valve Mfg. Co. viii
Turbine Equipment Co. xx
Wood, R. D., Co. xxvi
Worthington Pump and Machinery Corp. vii

RATE CONTROLLERS.

SHEAR GATES.
Mueller Co. xxxi

SLEEVES, PIPE LINE PEPAIR.
Dresser, S. R., Mfg. Co. ..... Following front cover

SLEEVES, RIVER.
Dresser, S. R., Mfg. Co. ..... Following front cover

SLEEVES AND VALVES, TAPPING.
Ludlow Valve Mfg. Co. xvii
Mueller Co. xxxi
Rensselaer Valve Co. xxiii
Smith Mfg. Co., The A. P. xvii

SLUICE GATES.
Mueller Co. xxxi

STANDPIPES.
Tredennick, J. H., Inc. xxv

SUPPLIES AND TOOLS.
Caldwell, George A., Co. xiii
Hays Mfg. Co. xviii
Leadite Co., The ..... Back cover
Pierce-Perry Co. xxv
Smith Mfg. Co., The A. P. xvii

TAPPING MACHINES.
Hays Mfg. Co. xviii
Mueller Co. xxxi
Red Hed Mfg. Co. xxxii
Smith Mfg. Co., The A. P. xvii

TAPPING SLEEVES. (See Sleeves and Valves, Tapping)

VALVE BOXES.
Bingham and Taylor iv
Caldwell, George A. Co. xiii
Ford Meter Box Co. xvii
Mueller Co. xxxi
Rensselaer Valve Co. xxiii
Wood, R. D., & Co. xxvi

VALVE INSERTING MACHINES.
Smith Mfg. Co., The A. P. xvii

VALVES, GATE.
Darling Valve & Mfg. Co. ix
Eddy Valve Co. xv
Kennedy Valve Mfg. Co. xiv
Mueller Co. xxxi
Rensselaer Valve Co. xxiii
Ross Valve Mfg. Co. viii
Smith Mfg. Co., The A. P. xvii
Wood, R. D., Co. xxvi

VALVES, REGULATING.
Mueller Co. xxxi
Ross Valve Mfg. Co. viii

WATER WASTE DETECTION.
Pitometer Co. ii

WATER WORKS INSTRUMENTS.
Caldwell, George A. Co. xiii

WATER WORKS JOURNALS.
Journal of the New England Water Works Association viii

WELL CONNECTIONS.
Red Hed Mfg. Co. xxxii

WROUGHT IRON PIPE. (See Pipe, Wrought Iron and Steel)

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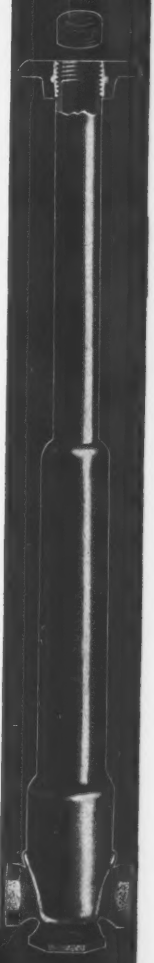

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H-10840
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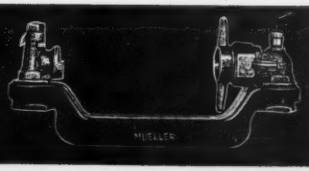

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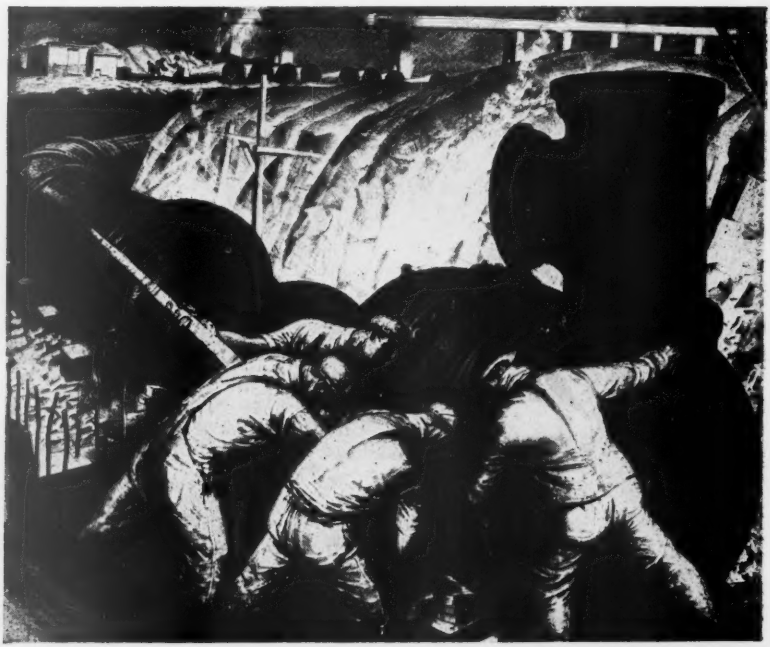

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